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THE STRUCTURAL ENGINEER

THE JOURNAL OF THE
INSTITUTION OF STRUCTURAL ENGINEERS

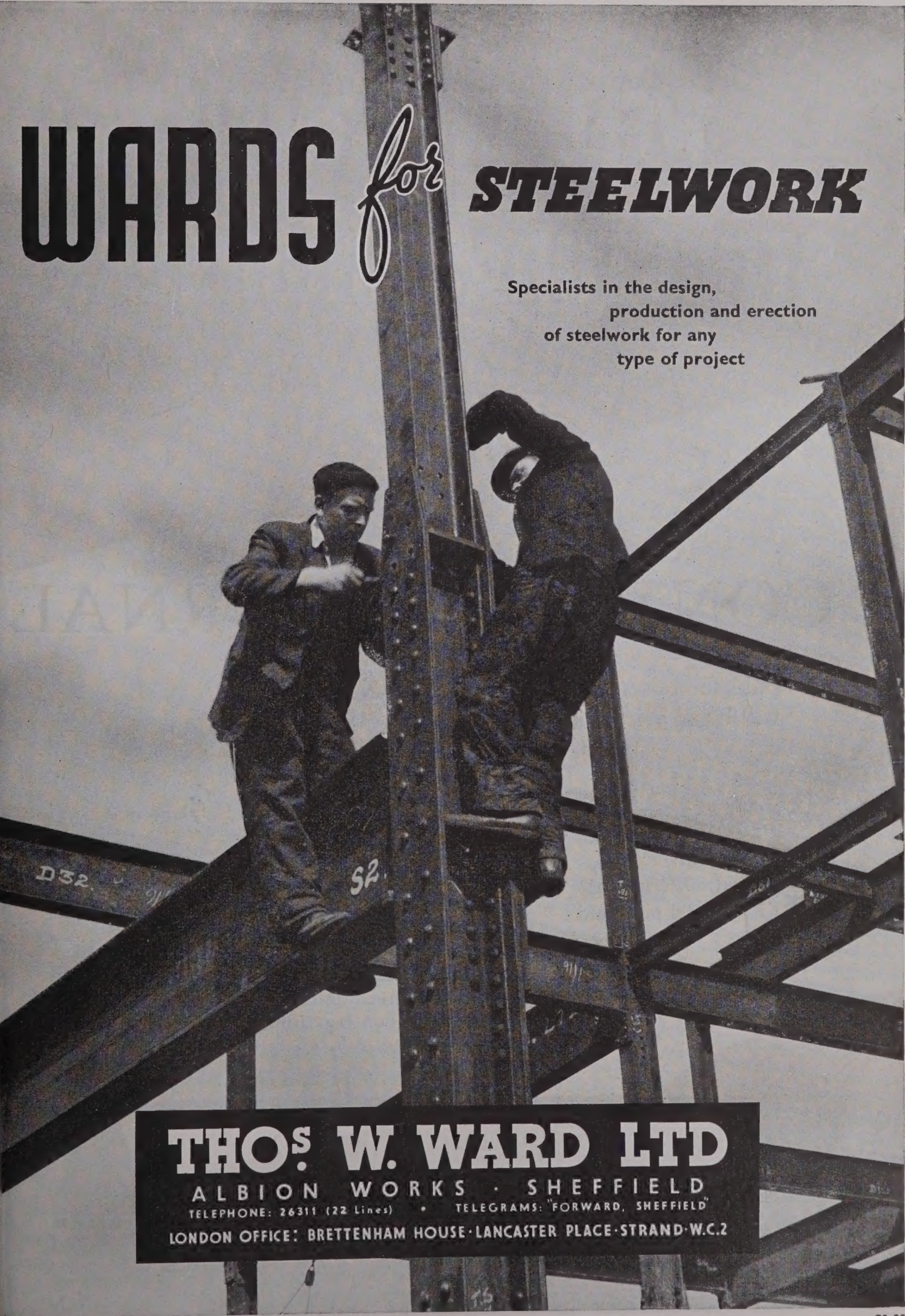
Presidential Address
by Lt.-Colonel G. W. Kirkland, M.B.E. (Mil.)

The 16 in. E.R.W. Tube Plant, Shotton
by W. T. Brooks, T. Burnett-Stuart (Graduate) and G. Bernard Godfrey (Associate Member)

Analysis of Braced Frames by Relaxation Method
by Professor S. L. Lee and R. E. Ball

Stiffening Girders of two Suspended Light Metal Footbridges at Alpnach, Switzerland
by Dr. K. Sutter and A. M. Mackie

FOUR SHILLINGS AND SIXPENCE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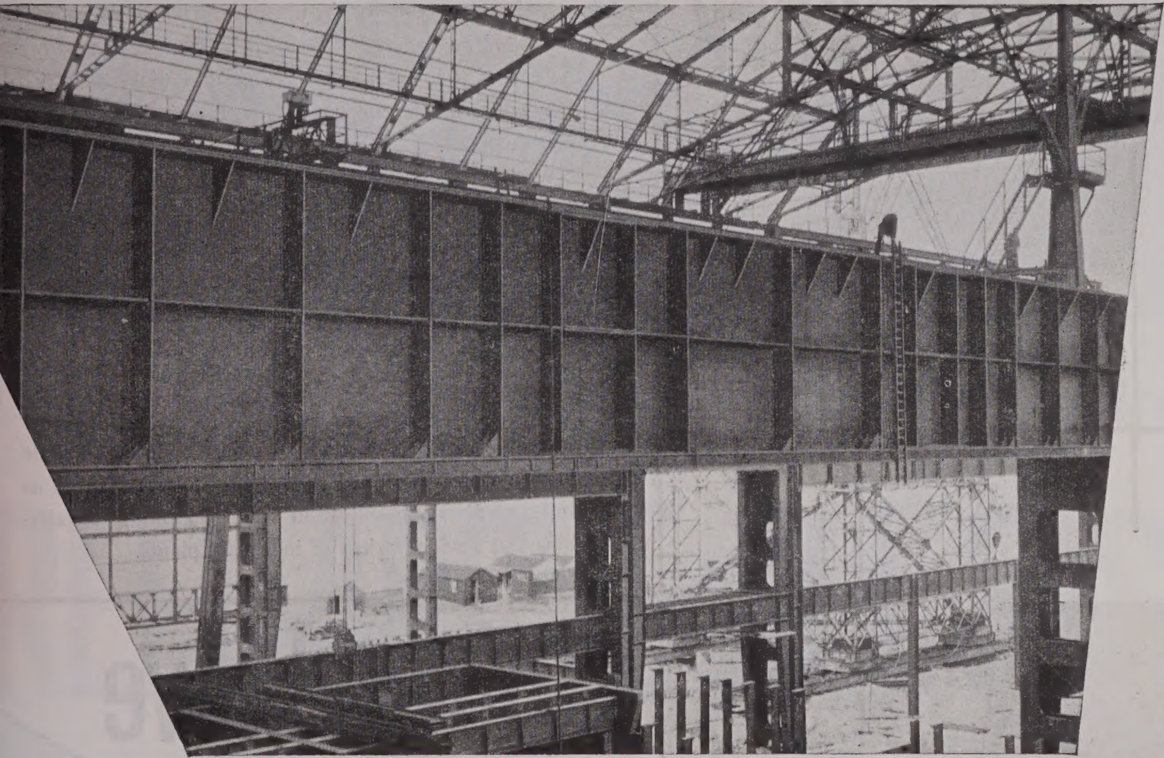

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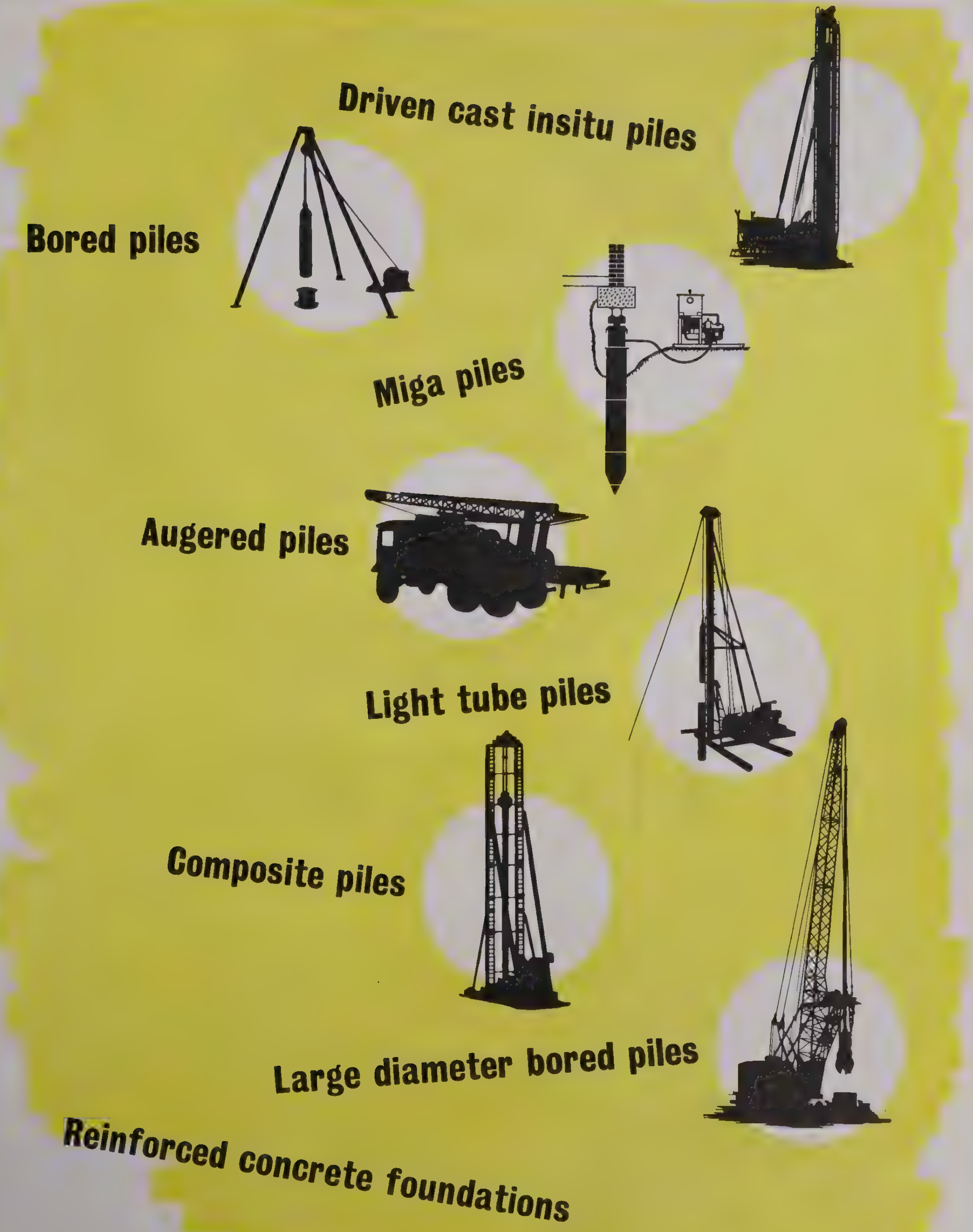

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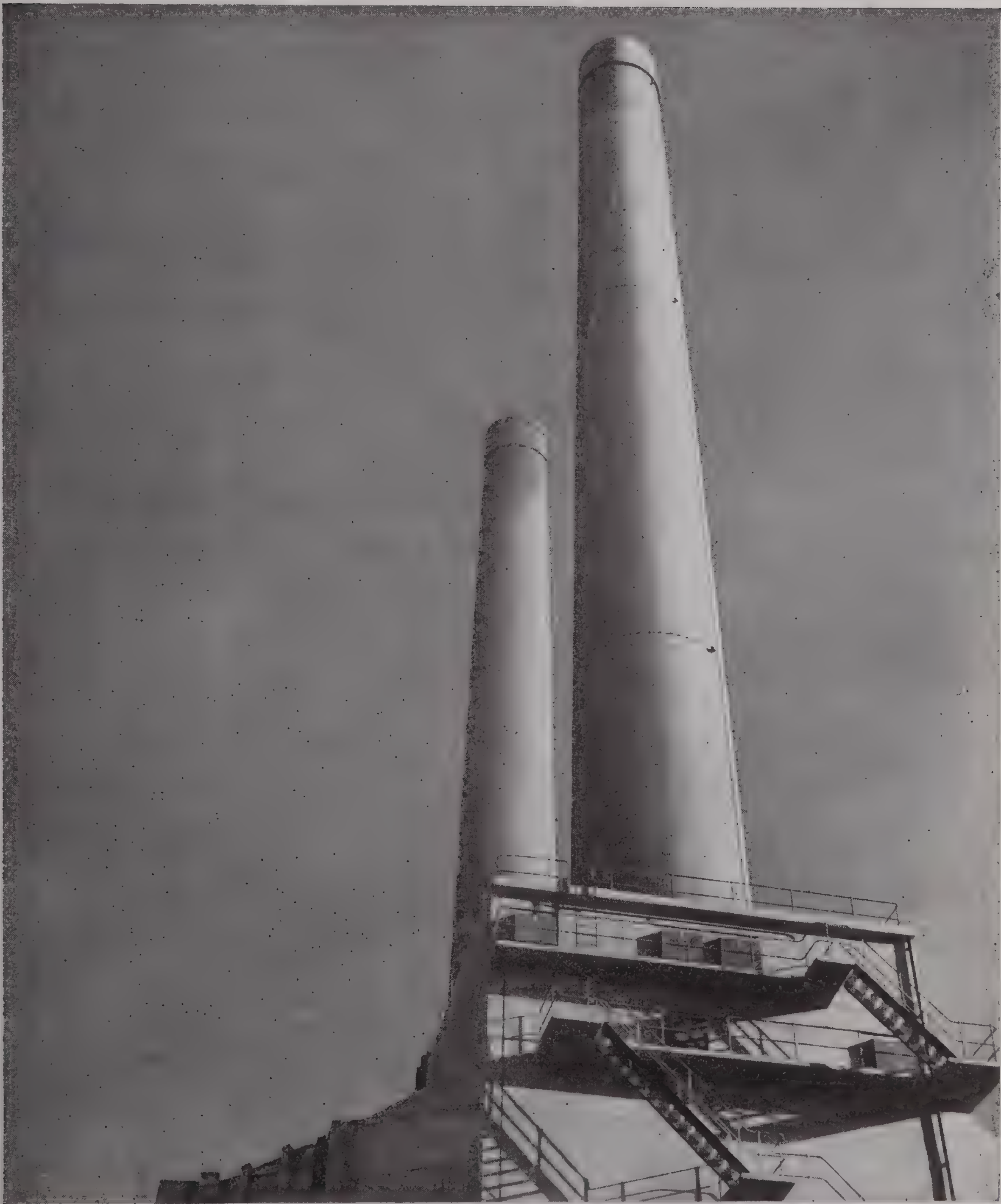


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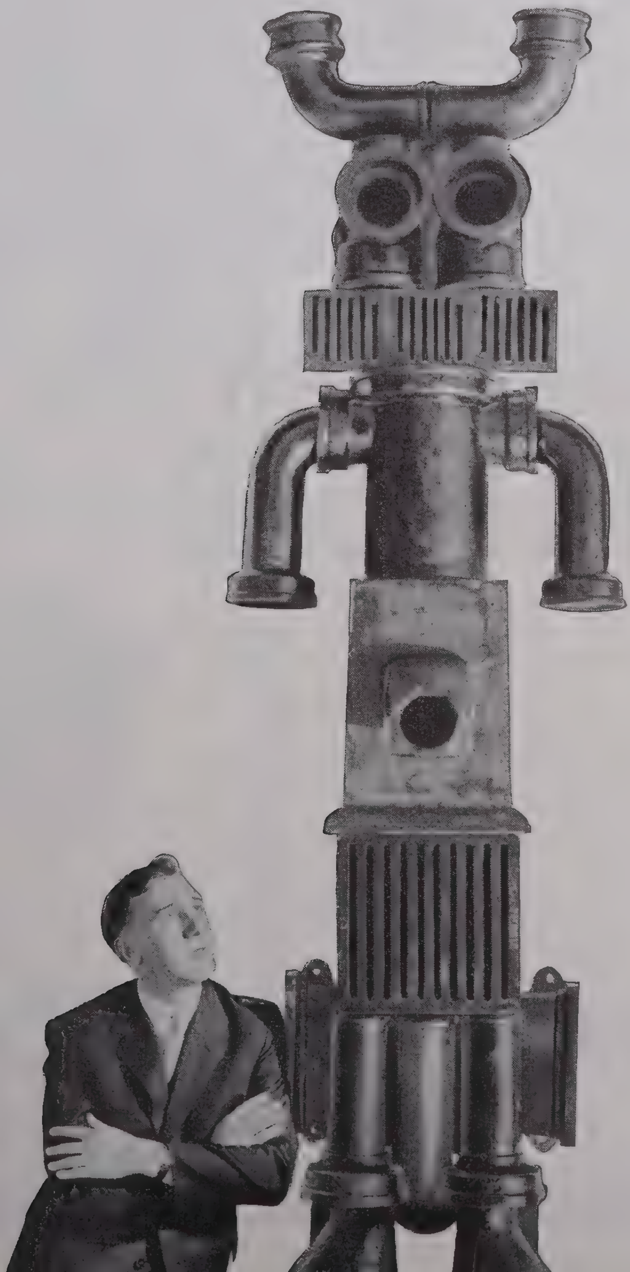

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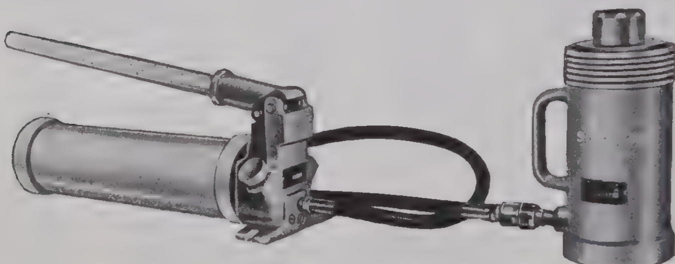

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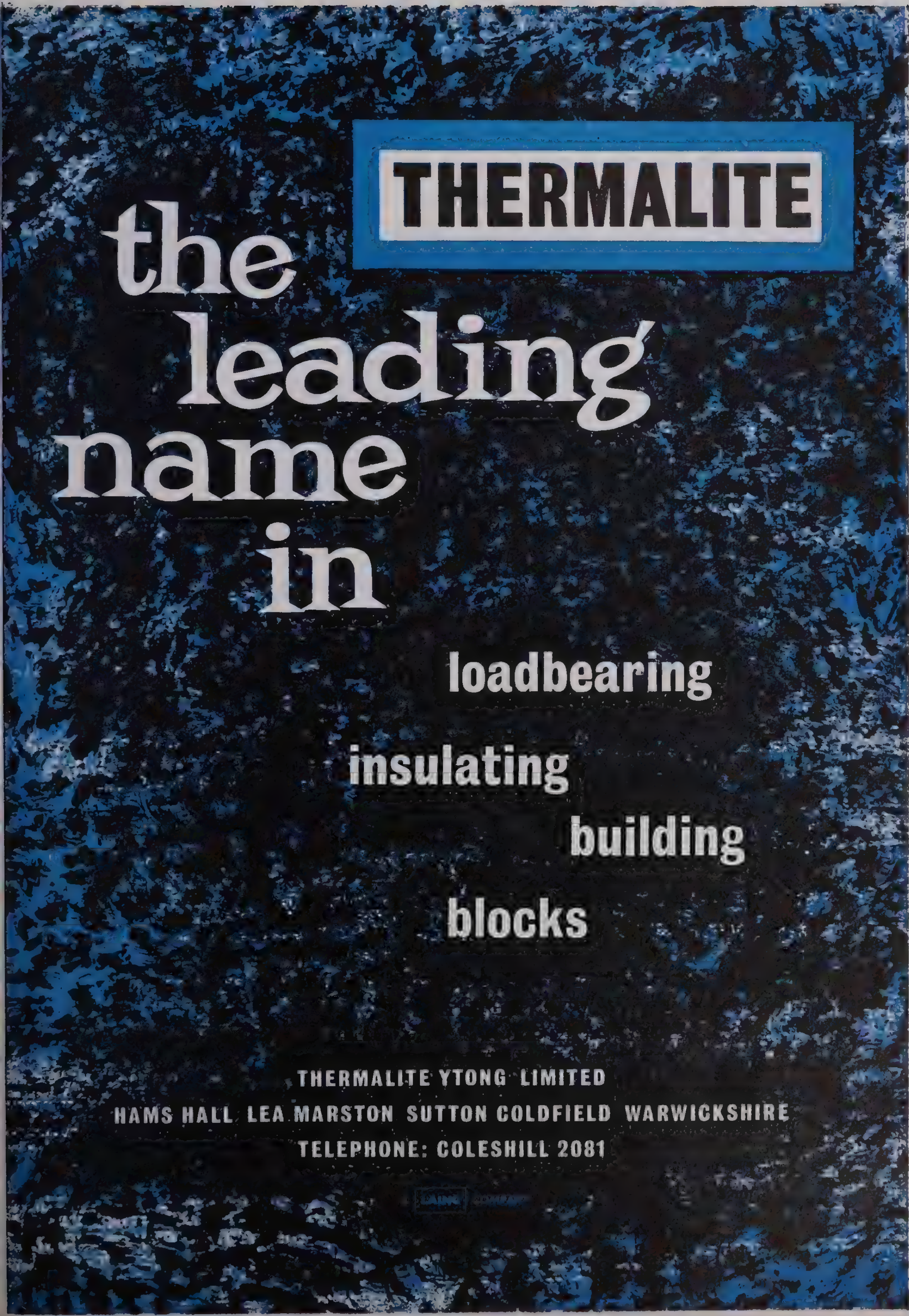
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CONTENTS

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PRESIDENTIAL ADDRESS

By Lt.-Colonel G. W. Kirkland, M.B.E.(Mil.), M.I.Struct.E., M.I.C.E.

Given before the Institution of Structural Engineers at 11, Upper Belgrave Street, London, S.W.1., on the 6th October, 1960

GENTLEMEN,

The Presidency of our great Institution, to which office you have elected me, has, in the past, been held by many distinguished engineers who, being dedicated men, have all assisted in increasing the stature of our body corporate. To say that I feel honoured in being allowed to join these august ranks would be a gross understatement, and yet those very sensations of honour and pleasure make me feel very humble.

In attaining this badge I am conscious of the many debts I owe to a number of people. Like my predecessor, I am going to name but three. They are, however, by no means all of those who have helped me to achieve this position and one of them was mentioned by Mr. Kent at the beginning of his session.

The first I shall name is one of our Past Presidents, Mr. Lawson Scott White. Mr. Scott White gave me my first appointment as his Chief Assistant and in following him he taught me much and gave me considerable encouragement.

The second one I name is a man of my own vintage and our immediate Past President, Mr. Lewis Kent. Our paths met long ago, even before I came to know Mr. Scott White. One might say we were boys together—at least he was! He was, and always has been, a man of so many interests and activities, he enjoys living and he has positively relished working for the Institution in no small measure. He is a very different man from myself—a sportsman of considerable merit in a number of fields, a man who had a highly successful Army career during that great period of emergency, but who always hides his light under a bushel in that connection. His successful year, only just concluded today, has been an example and inspiration which I shall do my best to follow.

Finally, our very great friend and 'pilot,' Major Maitland. No man could have been more dedicated to the Institution than has our Secretary over the course of many years and close association by any one of us with a man of this calibre could not fail but to leave its mark and, I believe, to have some of the spirit of dedication rub off on to one as a result of this association with him. None of us can quite aspire, however, to his almost Churchillian approach to his subject and his thoroughness in piloting the Institution and its Council through the various shoals which have been inevitable during the Institution's successful voyage, but with some of his spirit of dedication and our own goodwill we can but try to maintain the course which over the years he has set for us.

I would, at this juncture, especially like to add my very sincere thanks to all those past and present colleagues of mine on the Council. Their value cannot fail to be appreciated by those who follow us to this Chair, a course which takes us through the Chairmanship of certain Committees where the valuable contribution of each and every one of the Members of Council is fully appreciated.

When one is told by his colleagues that this Chair lies ahead, the Presidential Address looms large on one's horizon, both waking and sleeping. A story is told of the President—not of this Institution—who was dreaming he was delivering his Presidential Address and when he woke up he found that he was! One distinguished Past President stated only last May that the holder of this office must, if he is wise, start thinking seriously of the Address two years in advance of its presentation. I have been conscious of this task for some time. Four or five years ago, on a similar evening, I commented when proposing the vote of thanks to a newly installed President that the Presidential Address was almost a lottery since it was not pre-published and even its title was unknown until the moment of its presentation. You will appreciate, I know, my desire to avoid subjects which have been adequately covered in the recent past, and you will forgive me, I trust, if I show you how difficult a task this is. The field covered in the last few Presidential Addresses is as follows. Our immediate Past President addressed us on the subject of "What don't we know or What do we want to know." His predecessor, Mr. Macdonald entitled his Address "The Structural Engineer at Large," while Sir Alfred Pugsley spoke to us on "The Way of Research." Mr. Guthrie Brown gave us "Sixty Years of Hydro-Electric Development" (in Technicolour) and Mr. Stanley Vaughan "Twenty-one years of progress as a Chartered Institution." Dr. Hamilton chose "Developments of Structural Engineering as a Profession" and Colonel Galbraith spoke on "Modern Developments in Structural Engineering." "London's Bridges" was the choice of Mr. Granter, while Mr. Walter Andrews spoke on "The Developments of Structural Engineering as applied to Building." How rich a field is this in a mere nine years and then add to it that wealth of wisdom made available by all those who contributed to our Jubilee Conference. Then again, consider the quality of the first Maitland Medal Lecture and you will see the problem with which I was faced when selecting my subject. In addition to all this, I have been privileged in being allowed to present a number of papers to you myself, the most recent one in April this year. In the preparation of these papers, several thousand written words and much research have been involved in order to cover the subjects, and rather than run dry by trying to follow a purely technical field, I decided to try a new medicine on you, my colleagues.

My Address is an uneasy attempt at a philosophy and entitled "Equilibrium." Pythagoras is believed to have been the first to call himself a philosopher but I do not think it strange that an engineer should feel himself capable of expounding what he believes to be a modern application of an old philosophy. My reasoning may be faulty at times, my logic unsound, yet it is my hope that a germ of an idea will thrive despite the imperfections of the medium. Forgive me

then for a beginning which accepts a principle already known to Pythagoras. Many years ago now, when first starting my career, my Chief, finding me struggling with a simple problem, leaned over my shoulder and marked at the head of the paper on which I was working these symbols :

I was surprised, no doubt, and at the age of 16, symbols which were apparently unconnected with the matter in hand must have caused me to question the meaning, and my first lesson in engineering began. In reply to my query I was told the meaning and that it was that " the algebraic sum of the anti-clockwise forces must equal the algebraic sum of the clockwise forces if equilibrium is to be attained." I believe I was mesmerised by these words and fascinated by their meaning despite the fact that I was supposed to have learned the principle behind them as a schoolboy in my physics class. Gradually, my first problem seemed to fall into place and I sought every chance to try out my new understanding. Over the months that passed, other problems came my way and these symbols would appear, probably over a temporary absence from my place, over a lunch-time break or even overnight, placed there by my kindly Chief. Throughout my career I have found the principle so expressed of inestimable value and several of the young men who have worked with me have, I trust, similar recollections to those of my own on having seen these symbols appear on their own work.

As I progressed in the understanding of my chosen career, I passed through the phase of the triangle of forces, through that of chasing small arrows marked on the members of a braced frame to determine the stress state of the component parts, taking in the theorem of three moments and so on, appreciating at each step that early concept of equilibrium. During the course of the years, new tools have become available to the engineer for the solution of his problems. To the younger man, these tools are part of his equipment and his training has included teaching of their use and value. Names like those of Professor Hardy Cross, Professor J. F. Baker, Professor Southwell, and many others, are linked with these tools, and I am sure that an appreciation and understanding of them is more readily obtained with the basic conception that ultimate equilibrium is attained from their use. Over the course of years, my whole concept of equilibrium has deepened and it is with this thought in mind that I now wish to direct your thoughts to the extension of equilibrium.

The whole universe around us would seem to underline the importance of this theory of equilibrium, where star clusters and complex systems of matter are maintained in partial equilibrium, the balance being maintained by the creation of new masses or the destruction of old, whether one subscribes to the theory of expanding space or to the explosion theory of creation. The necessity of balancing mass against mass and force against force has resulted in many of the discoveries which have had so much effect on our civilization today. Einstein's Theory of Relativity was, I am sure, derived from the necessity of balancing the equation. All knowledge would appear to have derived from the necessity of maintaining equilibrium. In the time of Galileo the conception of the universe was based on a fixed earth with other stellar masses revolving around it. This view persisted for many centuries but theories of any kind must be bent or adjusted to comply with new facts and data, and the conception of the universe today is very different from that of the ancients, yet both in their own way attain the equilibrium so necessary for the peace of mind of those who concern themselves with these matters. In our earth itself the necessity for balance can be seen and the last twelve months with their crop of earthquakes in Chile and Africa and the recent outbreak of the volcano Mount Etna would seem to indicate how Nature's equilibrium is being maintained to give stability to the earth by allowing the ' safety valve ' action which produced these horrors. The daily papers seem nowadays to give almost continuous examples of the perpetual struggles of peoples, races and ideologies striving firstly to obtain their own equilibrium and subsequently to integrate that with the larger equilibrium of the world as a whole.

It seems to me that all professions have their own particular equilibrium. Law, its symbols the Scales of Justice and the Sword, is one of the easiest to appreciate. The equilibrium in her case lies between right and wrong, guilt and innocence, equity and injustice, and if the scales be loaded one side it might well be said that equilibrium is maintained by the weight of evidence. The factor of safety can err in law but seldom to the prejudice of the individual.

As with Law, so too with Accountancy. Equilibrium must be maintained between income and expenditure although this learned profession sometimes shows us means these days of maintaining financial equilibrium when it seems nigh impossible. The legislation covering our country's finances would appear, to my mind, to have been planned over the course of years with the object of raising the standards of living of the less fortunate ones of this country while ensuring that there was no excess income received at the other end of the scale. We may feel that this has involved hardships in some cases, particularly with the professional engineer. Yet the necessity to equilibrate income and expenditure in developing firms in the steel industry, in the cement industry, and even in professional practice, seems to continue to be possible with the help of our accountants. I like the story which tells of the successful firm entering its tenth year of operation with an overdraft of some £25,000. The Directors were most concerned and asked their accountants' views on the dangers resulting from this. They were told that there was no doubt about their success in their business since only three years previously the bank had applied a restrictive limit to their borrowing of a mere £500.

With Architecture I had some difficulty in finding this relation to equilibrium. In the sense of pure construction it seemed that the engineer's equilibrium was the one which was applicable but if one accepts Architecture as a professional art, one can see the necessity for equating the economics of construction with the appearance of the finished structure. The Architect must plough in all his effort and desire in expressing himself and his patron's needs in terms of building and despite much criticism I am certain that a goal will be reached in time where this generation will not be shown to have been devoid of endeavour. There will, of course, always be the monotonous box, the attempt to caricature, the personal monument, and like methods of expression that have not achieved the greatest success, but they are, without a doubt, all efforts towards an end which fortunately advances

beyond the generation trying to achieve it. The ambition and endeavour of the profession is then maintained in equilibrium by the effort to create something better than that which was done before.

Similarly, with Medicine, certainly so far as the treatment of disease is concerned. The human equilibrium between virus and bacillary invasion and antibody resistance appeals to me as medicine's equal and opposite forces and our calls on medical aid when indisposed are merely an endeavour to balance this eternal equation. It has been contended that a very large part of the human ailments are psychological in origin and one wonders to what extent the individual's happiness can affect the degree of incidence of illness. Carl Jung, that great old philosopher, now 85 years of age, was recently interviewed for the Sunday Press. Jung, a philosopher of considerable merit, has, I am sure, a number of valuable contributions still to make. From the report of this interview, I quote Dr. Jung's five basic factors for happiness in the human mind :

1. Good physical and mental health.
2. Good personal and intimate relations.
3. The faculty for perceiving beauty in art and nature.
4. Reasonable standards of living and satisfactory work.
5. A philosophic or religious point of view capable of coping successfully with the vicissitudes of life.

If our mental health can affect our physical condition, there is a sense of incomplete circle in these five elements, to my mind, but I see the completion of the equation in one's ability to put into life those efforts which are necessary to balance the subordinate equations, both in terms of one's profession, one's immediate society, and friendships.

In terms of our immediate society, we think of Engineering and it has been stated many times that engineers speak with too many voices. I think it is quite true that people as a whole do not appreciate the vast contributions which engineers have made to civilisation and it is my feeling that our contributions have almost entirely been constructive in a world which seems, to my mind, slightly self-destroying at the moment. Is then our contribution to human equilibrium sufficient ? In what way can we overcome this inequality? There are, I believe, some thirty or more engineering bodies, some chartered, some merely incorporated, each having their own field of activity and endeavouring to speak for themselves in terms of national effort. The formation of these various bodies has been an obvious necessity and the creation of new bodies as science advances would seem, to my mind, to be completely justified and yet, by their very creation, the engineer as a whole becomes less well represented. I have a theory in this connection that in the course of time there will be one body to speak for all engineers—a non-examining body maybe—the control of which will be by a Council on which all the engineering associations and institutions would be represented. The impact of several hundred thousand engineer/scientists speaking through such a body to either government or people or to their professions could not fail to be profound.

I hope that up to this stage you have followed my attempt at giving you the thread of equilibrium and I would now like to turn to our own Institution. Under the charter our aims are specified that we stand for the advancement of the science and art of structural engineering. The acquisition of knowledge is a gradual and intriguing process and, as I have tried to indicate earlier, the balancing of the equation can only be attained by the solution of the new problems which resulted from the previous process of obtaining equilibrium. Despite the incompleteness of each stage, an advance is always made in that the knowledge and experience which have been gained by us can become available for the enlightenment of others so that they may start a little further advanced than we, in our generation, have done, and so the future can be assisted by reducing the journey which the newcomer must travel.

I have referred to the devotion of our Presidents, our Secretary, our Members of Council and so many of those associated with us and I may say the evidence of their dedication to the aims of the Institution can be seen in the progress we have made. But all of you here, Members of this body, have obtained from the Institution some reward or look to that reward, in due time, of corporate membership. Doesn't it occur to you that your personal equilibrium can be enhanced by your endeavour to contribute something to the attainment of these very aims ?

One point which struck me many years ago, when acting as a scrutineer, was the few members who bothered to record a vote and I do hope that these words will reach some of you who have not voted in the past. While membership continues to increase yearly, it is always to me a matter of interest to see the percentage of the electorate which does vote. We complain, many of us I fear, of indifference in so many phases of present-day life ; in the political field, the various parties keenly follow the proportions of the voting electorate who do contribute to the ballot ; in the Trades Unions we, many of us, criticise the lack of individual interest in matters which affect the entire life and structure of the body which they represent, and even in religious circles we have the same feeling. And yet we would seem to be no better than so many of the others in this very same respect. Our Institution is to us the major part of the society in which we work. The same may be said of the other professional bodies in relation to the society which forms their membership and I do most sincerely hope that the future will show an ever increasing interest in the activities of this Institution in all its fields.

Finally, I propose making this an opportunity for a special appeal to you in terms of the Institution's Benevolent Fund. This fund, I feel, was created out of the desire of its founders to maintain the equilibrium of our society by taking care of those less fortunate than ourselves. I know that many appeals are made on all of us these days to help this or that body and to contribute to causes of so many kinds. May I suggest, however, that for us structural engineers our own Benevolent Fund would make a good beginning and may I ask you if you could contribute as a clockwise force in terms of a covenanted subscription to counteract the anticlockwise forces of those less fortunate than yourselves. It may well be that should the organisation of a central body, non-examining, to speak for engineers ever be realised, a combined benevolent fund may well supersede our own but this time is many years ahead and our needs are for the present.

It has, for me, been a wonderful day at the foot of the ladder which I must climb through the next twelve months. I shall be critical of myself and I may be, on occasion, of others, but I do want you to know that the interests of the Institution will be the standard from which my criticisms may spring and it will be with the interest of the Institution at heart for the benefit of the science and art in which we are all occupied that I shall strive.

The 16 in. E.R.W. Tube Plant, Shotton*

by W. T. Brooks, T. Burnett-Stuart (Graduate)
and G. Bernard Godfrey, A.M.I.Struct.E., A.M.I.C.E., A.M.I.Mun.E.

INTRODUCTION

The object of this paper is to describe the structural aspects of the plant erected at Shotton, Flintshire, in which Stewarts and Lloyds will produce welded oil-line pipe up to 16 in. outside diameter by the Electric Resistance Welded (E.R.W.) process.

Since the capacity of the proposed plant was some 200 to 300 thousand tons per annum and the market for the product lay mostly abroad, it was essential that the site selected should be between the raw material and an appropriate port. The site finally chosen in 1957 was just to the south-east of the integrated steelworks of John Summers & Sons, with whom an agreement was made for the supply of the appropriate coiled hot rolled strip. In consequence, the raw material may be brought to the new plant by short haul road transport and the finished product dispatched by road or rail to such ports as Liverpool, Manchester and Ellesmere Port, within 35 miles of the site.

At an early stage, Stewarts and Lloyds prepared preliminary specifications for the buildings and other structures necessary to house the mill, finishing plant and ancillary services, together with plant layouts and ground loading diagrams for the plant foundations.

For the foundations and civil engineering work, the preliminary documents were sent out to seventeen civil engineering contractors who submitted price schedules and target prices for the complete job based on preliminary bills of quantities. Comparisons of the quotations received resulted in the appointment of Holst & Co., who included for all building and plant foundation work with all the detail design associated with it. This selection was additionally fortunate as these contractors had exceptional local knowledge and a suitable organisation already set up in the adjoining works of John Summers.

For the structures, enquiries were issued to six firms for designs based on several specifications which comprised complete tubular construction, partial tubular construction and construction entirely in rolled sections, with alternative roof designs and insulation methods as further variants. After receipt of these quotations and comparison of the different designs and prices, Tubewrights were appointed to supply the complete structures. This decision was based not only on the price being competitive, but also the fact that it was the only design put forward for almost entirely tubular construction.

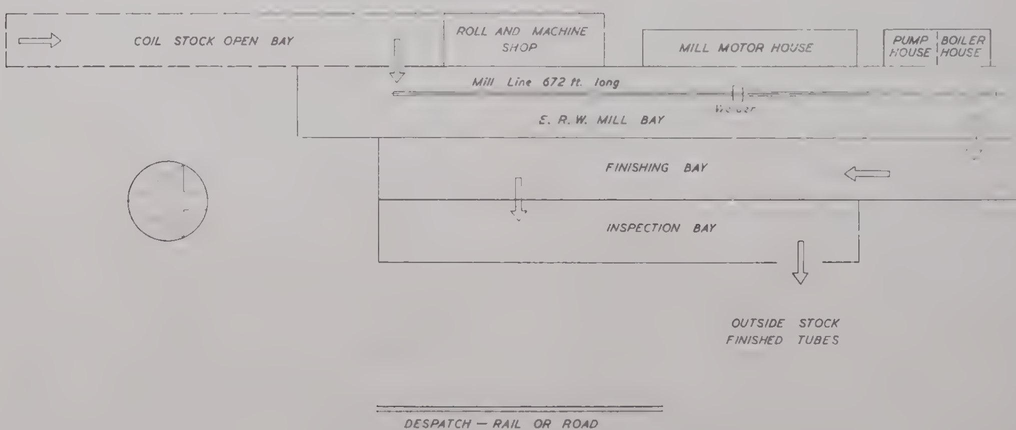

Fig. 1.—Layout of plant.

The dimensions of the various structures were dictated by the geometry of the plant. Fig. 1 gives a plan of the various structures required and shows the flow through the plant while Table 1 gives other salient data.

In the case of the main mill in particular, careful consideration was given to plant clearance heights and crane lifts, while adequate space was essential in the width of the bay, not only for the plant and its drives, but also for access and the storage of the large quantities of tools associated with the mill.

THE E.R.W. PROCESS

As it is probable that comparatively few readers will have had the opportunity of visiting an E.R.W. mill, the various rather picturesquely-named stages of manufacture are shown diagrammatically in Fig. 2,

**Paper to be read before the Institution of Structural Engineers at 11, Upper Belgrave Street, London, S.W.1, on Thursday, 10th November, 1960, at 6 p.m.*

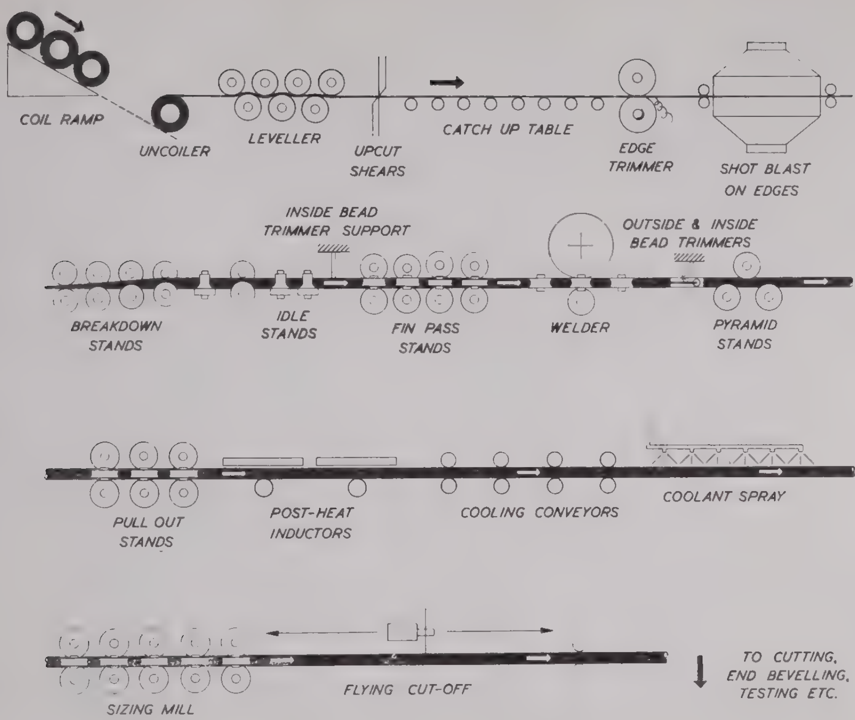

Fig. 2.—The E.R.W. process.

The actual process is carried out in one straight line and as it is essential that there should be no deformation of the tube during production, no settlement of the mill foundations could be entertained.

FOUNDATIONS

Site Conditions

As most steel processes require long buildings, it is advantageous to have a large level site. The site chosen, shown in its original state in Fig. 3 and euphemistically referred to in the trade as a green field site, lay in the angle between the Sealand-Wrexham railway line which runs roughly north to south, and the main access road to John Summers' works. A detailed survey showed that the average ground level was + 13·30 O.D. This area became flooded in winter after prolonged heavy rain to a level of approximately + 15·00 O.D. Many hundreds of acres drained into channels, known locally as ' gutters,' with only one outlet to the River Dee, which passed under the railway line.

Fig. 3.—The original site.

TABLE 1. Specification for Structures

Reference	*A*	*B*	*C*	*D*	*E*	*F*	*G*	*K*
Building	E.R.W. Mill Bay	Finishing Bay	Inspection Bay	Roll and Machine Shop	Mill Motor House	Pump House	Boiler House	Coil Stock Open Bay
Span	80 ft.	70 ft.	70 ft.	60 ft.	40 ft.	40 ft.	40 ft.	60 ft.
Length	810 ft.	720 ft.	540 ft.	180 ft.	240 ft.	60 ft.	60 ft.	495 ft.
Type of Roof	Northlight construction	Northlight construction	Northlight construction	Northlight construction	Flat roof	Northlight construction	Northlight construction	
Main Column Centres	East Side 30′0″ and 45′0″	East Side 45′0″ and one at 90′0″	East Side 45′0″ and 60′0″	30 ft.	30 ft.	30 ft.	30 ft.	30 ft. and 45 ft.
	West Side 45′0″ and one at 90′0″	West Side 45′0″ and 60′0″	West Side 45′0″ and 60′0″					
Crane Rail Crs.	75 ft.	66 ft.	66 ft.	56 ft.	36 ft.			56 ft.
Height—Crane rail to floor level	28′0″	22′0″	22′0″	22′0″	16′0″			22′0″
Height—Crane Headroom	9′0″	7′0″	7′0″	7′0″	4′0″			
Height to bottom C.L. of truss	38′0″	30′0″	30′0″	30′0″	21′0″ to underside of roof	25′0″	25′0″	
Cranes, number and capacity	One, 20 ton, E.O.T.	One, 5 ton, E O.T.	One, 5 ton, E.O.T.	One, 5 ton, E.O.T.	One, 3 ton, hand operated.		No crane	Two. 15 ton. E.O T.
Roof trusses type and centres	Tubular at 22′6″ Crs.	Tubular at 22′6″ Crs.	Tubular at 22′6″ Crs.	Tubular at 15′0″ Crs.	Castella beams at 30′0″ Crs. with R.S.J. purlins.	Tubular at 15′0″ Crs.	Tubular at 15′0″ Crs.	
Side Walls	East side 4′0″ wall and R.P.M. sheeting at coil stock and boiler house ends, and between buildings Ref. D & E, and E & F and above the adjacent buildings. West side 4′0″ wall and R.P.M. sheeting at north end, remainder of west side open to finishing bay Ref. B.	East side open to mill bay Ref. A. West side 4′0″ and R.P.M. sheeting at south end, remainder of west side open to inspection bay Ref. C.	East side open to finishing bay Ref. B. West side 4′0″ wall and R.P.M. sheeting with two tube discharge openings at south end.	East side 4′0″ wall and R.P.M. sheeting.	East and west sides bricked to eaves with two industrial windows in east side and with air conditioning plant lean-to and transformer bays attached to east side.	East and west sides 4′0″ wall, with R.P.M. sheeting with three industrial windows in east side.	Same as Bay Ref. F.	

nd Walls	North and south ends each 4′0″ walls and R.P.M. sheeting to ridge of north end and valley of south end.	Same as Bay Ref. A.	Same as Bay Ref. A.	Same as Bay Ref. A.	North end bricked to eaves with 4 industrial windows. South end bricked to eaves with 4 industrial windows.	North end 4′0″ wall with R.P.M. sheeting. South end 4′0″ wall and R.P.M. sheeting above to form dividing wall between bays Ref. F & G.	North end 4′0″ wall, and R.P.M. sheeting above to form dividing wall between bays Ref. F & G. South end 4′0″ wall with R.P.M. sheeting.
nsulation	Brick walls 11″ thick cavity type with loose top tiles and necessary ventilation bricks, the roof and sides lined with $\frac{1}{2}$″ Celotex flame retardant insulation board between sheeting and steelwork.	Same as Bay Ref. A.	Same as Bay Ref. A.	Same as Bay Ref. A.	Brick walls 11″ thick cavity type necessary ventilation bricks and flame retardant insulation board to roof.	Same as Bay Ref. A.	Same as Bay Ref. A.
oofing Material	R.P.M. sheeting and glazing to vertical face of each ridge.	Same as Bay Ref. A.	Same as Bay Ref. A.	Same as Bay Ref. A.	Briggs 'Bitumetal' decking.	Same as Bay Ref. A.	Same as Bay Ref. A.
lazing	Patent glazing with $\frac{1}{4}$″ wired glass and 4 lb. lead flashing. Area of glazing not less than $33\frac{1}{3}$ per cent of the bay floor area.	Same as Bay Ref. A.	Same as Bay Ref. A.	Same as Bay Ref. A.	East side, one type SSF44 and one type SS53 industrial window. North end, 4 type SS53 industrial window. South end, 4 type SS54 industrial window.	Patent glazing to roof with $\frac{1}{4}$″ wired glass and 4 lb. lead flashing ; area of roof glazing not less than $33\frac{1}{3}$ per cent of the bay floor area. East side, three type SSF54 industrial windows.	Patent glazing to roof with $\frac{1}{4}$″ wired glass and 4 lb. lead flashing ; area of roof glazing not less than $33\frac{1}{3}$ per cent of the bay floor area. East side, three type SSF54 industrial windows.
rovision for Extension	At south end.	At both north and south ends.	At both north and south ends.	None.	None.	None.	None.
oors	East side, one D/L rubber door opening 9′0″ wide × 10′0″ high, three S/L timber doors opening 2′9″ × 6′6″ high. North end, one roller shutter door opening 24′0″ wide × 14′6″ high, one S/L timber door opening 2′9″ × 6′6″ high. South end, one S/L timber door opening 2′9″ × 6′6″ high.	North end, one roller shutter door opening 16′0″ wide × 14′6″ high, one S/L timber door opening 2′9″ × 6′6″ high. West side, one S/L timber door opening 2′9″ × 6′6″ high.	North end, one roller shutter door opening 16′0″ wide × 14′6″ high. West side, two S/L timber doors opening 2′9″ × 6′6″ high. South end, one S/L timber door opening 2′9″ × 6′6″ high.	North end, one S/L timber door opening 2′9″ × 6′6″ high. East side, one roller shutter door opening 14′0″ · 14′6″ high. West side, one opening 14′0″ wide × 14′6″ high near mid-length, D/L sliding door.	East side, one D/L timber door opening 10′0″ wide × 10′0″ high, one roller shutter door opening 14′0″ wide × 14′6″ high. West side, one D/L timber door opening 10′0″ wide × 10′0″ high, four S/L timber doors opening 2′9″ × 6′6″ high. Internal store, one S/L timber door opening 2′9″ × 6′6″ high.	East side, one roller shutter door opening 14′0″ wide × 14′6″ high. West side, one S/L timber door opening 2′9″ × 6′6″ high.	East side, one roller shutter door opening 14′0″ wide × 14′6″ high. West side, one S/L timber door opening 2′9″ × 6′6″ high.

The River Dee was some one and a half miles from the site, as will be seen in the aerial view in Fig. 4 which also shows the partly-completed plant in the left foreground, and the head in the main gutter was minute. It was, therefore, decided to install a small pumping station on the west side of the railway embankment to reduce the water level over the site to + 11·00 O.D., there being a few days' time lag before this level was reached after prolonged heavy rain.

A knowledge of conditions in the construction of foundations in this type of ground had been built up over many years in the adjacent steelworks. A borehole, driven some years ago, seven hundred yards to the west of this particular site, showed the ground to be made up of one foot of top soil and 180 ft. of fine clean sand, with a few thin layers of gravel at varying intervals, below which there was rock. Well borings about one mile east of the site showed the same kind of sand and gravel with rock at a depth of 90 ft. Hence, by interpolation, it was assumed that rock lay about 150 ft. below the site.

The floor level of the tube mill was tentatively fixed at + 22·50 O.D., just over 9 ft. higher than the average ground level. The filling used over the whole area of the site was sand, identical with that existing below top soil, which was pump dredged from the River Dee. This sand is reputed to be the finest graded river sand known.

Before dredging started it was necessary to remove all peat from the gutters and strip completely all top soil for three main reasons :—

(a) The top soil was easily compressible and would allow settlement of any loaded, unpiled structure or floor.

(b) It was necessary to wellpoint certain of the deeper mill installations extending below ground water level and the existence of water above the impervious top soil and peat would have caused water to flow through the embankment of any wellpointed area at top soil and peat levels. This could have been off-set by the use of sheet piles instead of open excavation embankments, but this method is expensive and previous experience had shown that steel sheet piles driven in this ground were very difficult to extract, it being cheaper to leave them in.

In addition to this, it was necessary to take into account the extra expense of impeded work in heavily strutted holes. In these ground conditions steel sheet pile cofferdams could not be used without using wellpoints, as the ground at the bottom of the hole was unstable and the sand would have flowed into the cofferdam with the water, tending to reach a level of + 11·00 O.D.

(c) The top soil was required to stabilise the embankments of the filling placed at a slope of $1:1\frac{1}{2}$ and to form lawns on the top of the filling where these were required.

As the filling area completely blocked the ground gutters draining water from the marsh area to the culvert under the railway embankment, it was necessary to surround all the filling with a wide ditch at the bottom of the embankments. Although at

Fig. 4.—Aerial view of the site.

a later date the pumping station reduced the water in these ditches to the + 11·00 O.D. level, the water level in the filling remained at approximately + 14·00 O.D. and it was necessary to wellpoint any excavations below this level.

Access Roads

Access roads on the site were made by laying a 12 in. thickness of blast-furnace slag to the required camber and compacting with a heavy vibrating roller. Where possible, the layers of slag were placed along the line of permanent roads to form a well-compacted base to the finished road surfaces which were laid on completion of the other constructional work.

General Excavation

General excavation and trenching above water level was carried out by the open-cut method as timbered excavations needed to be almost water-tight to prevent the sand running through the joints of the timber, a fact which could have caused expensive delays where reinforcement had to be laid in the bottom of an excavation.

It was not economical to drive interlocking steel trench sheets, as they would not penetrate further than five feet when driven by hand.

Before starting any excavations, check levels were taken over the surface of the filling and after making preliminary cut and fill calculations it was decided that it would be more economical if the mill floor level were lifted 6 in. to a level of + 23·00 O.D. This obviated the necessity of carting approximately 4,500 cu. yds. of excavated material to tip.

Piling

As bored piles had not proved satisfactory in this type of ground, the piles used were 12 in. square of precast concrete, with central chisel points formed in concrete without a metal shoe. They were cast on an existing pile bed about one mile from the site and were carted there by heavy road transport.

The piles were driven by drop hammer to the required set and from previous experience it was known that those driven from fill level would not penetrate lower than —2·00 O.D. and there would be up to 4 ft. less penetration for many of the piles, according to the tightness of the pile groups driven. The largest pile used was 25 ft. long and the average length of all piles driven was 22 ft. The piles which penetrated to the greatest depth were those driven over the old drainage gutters in the original marsh under the filling.

In many cases, the mill foundations immediately adjacent to column foundations were taken down to a depth below ground water level and the excavations wellpointed, in one instance a depth as low as + 2·00 O.D. being reached, approximately 12 ft. below site water level. In these cases, the piles were driven from the lower wellpointed formation.

Pile Loading

Near the previously-mentioned borehole, 700 yds. west of the site, a 14 in. square pile has recently been driven from a filling level of + 28·00 O.D. for a depth of 23·8 ft. to a ¼ in. set, obtained with ten blows of a three-ton hammer dropped from a height of 30 in. Using a load factor of 2·5, the revised Hiley formula gave a *working load of* 124 tons. When a load of 130 tons was applied to the pile for 24 hours the head depressed 0·120 in. When the load was removed, the pile head reasserted itself to its original position, showing no residual settlement.

When a pile is driven into this particular ground, there is no displacement of the ground at surface level. Once a satisfactory set has been obtained on the pile, the ground resistance is practically permanent. Over the past 20 years, many thousands of piles have been driven in the same marsh area and no settlement has yet been recorded.

Although there is no displacement at the surface when such a pile is driven, displacement equal to the volume of the pile occurs in the ground. The void content of the sand is reduced and the sand is compacted almost to the hardness of a good sand-stone. This has been proved by excavating around driven piles to inspect the condition of the sand near the toe of the pile. In effect, the driving of precast piles is a form of ground reconstitution. If 12 in. square piles are driven in a group at three feet centres, the void content of the sand is reduced by 11 per cent.

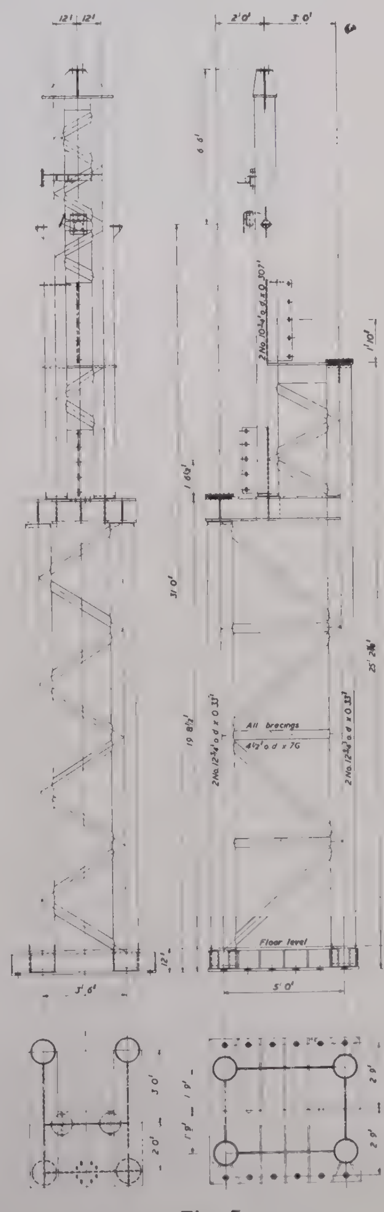

Fig. 5.

The piles have considerable resistance to uplift and tests have shown that no movement of the pile has occurred when a tension of 15 tons has been applied to the head of the pile for a period of three days, nor has uplift occurred when a 15 ton lift was applied by snatch loading, every 20 seconds, for a period of 30 minutes.

Pile Layout and Cap Design

The figures for loading and the values of the overturning moments were supplied by the steelwork contractors. These data comprised all possible cases of loading to the columns, the worst possible conditions being used to design the foundations.

Of particular interest are two pairs of very stiff columns, situated between the main mill (A) and the finishing bay (B). As it was not possible to use the conventional portal bracing, these columns, details of which are shown in Fig. 5, were designed to resist the longitudinal surge and wind forces as individual vertical cantilevers.

Many of the combinations of loads were eliminated by inspection and eight combinations only were used in the final design. The worst conditions were as follows :—

1. Maximum dead load—173 tons.
2. Maximum overturning moment—612 tons ft.
3. Maximum eccentricity of any dead load due to overturning moment —4·73 ft.

The maximum dead load was obtained when the E.O.T. cranes in the adjoining bays were carrying maximum loads, the loads being near the columns and the cranes directly over the columns.

The maximum overturning moment was obtained by taking transverse surge from one side only with longitudinal wind pressure in addition to longitudinal surge from the crane in the adjacent bay.

The maximum eccentricity of 4·73 ft. was obtained when overturning moments were applied with a dead load of only 23 tons. This distance was of little consequence, as the dead load was small, compared with an eccentricity of say, 1 ft., when the maximum dead load of 173 tons was applied.

Ten piles were used for these columns, compared with five for adjacent columns, as shown in Fig. 6.

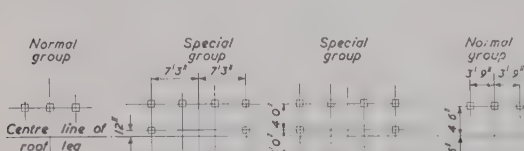

Fig. 6.—Layout of piles.

The maximum load on any pile amounted to 41·5 tons and the maximum uplift on any pile was 2·2 tons, the latter being produced when full wind loading was applied with the dead load of the structure only.

To reduce the uplift to a minimum, the pile cap was designed to be 6 ft. thick with a total weight of 86 tons. An alternative method of reducing uplift would have been to increase the plan area of the pile group, thereby increasing the inertia of the whole group and the size of the cap. However, such a cap would project well into the area of the shop floor where it could be subjected to casual but very heavy loading for which it would be uneconomical to make allowance.

In general, the design should be an economical balance of the disposition and number of piles and the overall size of the cap. Such a design is achieved by trial and error. The first attempt is made from experience and checked with the various combinations of loading. Should the first design be unsatisfactory, adjustments are made until a suitable layout is achieved. The pile group usually evolves from the eccentricity of the principal dead loads and the eccentricity caused by the overturning moments.

In the particular design being considered, the longitudinal moments applied to the column (i.e. the moments obtained by horizontal forces acting in the direction of the crane rails) were the same in both directions and the column could be concentric on the pile group in this direction.

As is usual, there was eccentricity in the transverse direction due to crane loads and transverse surge and wind forces. The lever arm for these forces was calculated about the underside of the pile cap and not about the base of the steel column. The eccentricities were separately calculated for each direction of moment, with the result that in two combinations the maximum dead load could be 0·6 ft. eccentric to the west or 2·5 ft. eccentric to the east. It was finally decided to place the column eccentric to the cap by 1 ft. to the west of the cap centre-line.

The pile loads were then checked for each combination of loading, the corner piles usually being the most heavily loaded. The summation of loads on any one pile comprised :

1. Total dead load at the foot of the column plus the total weight of the pile cap, divided by the number of piles.
2. Induced load on the piles from the mathematical eccentricity of the column due to the resultant eccentricity of the vertical loads, together with eccentricity caused by horizontal forces from the transverse and longitudinal directions, plus or minus the physical eccentricity of the centre-line of the cap about the centre-line of the column.

The shear on the pile cap, produced by the horizontal forces on the column, was resisted by passive ground resistance or by floor slabs, where these were available.

Of the more usual columns elsewhere in the plant, mention may be made of those on the east side of the main mill (A), shown on the right in Fig. 7.

The main overturning moments act inwardly as the 20 ton shop crane is carried on the inside leg of the column only. The eccentricity of the main dead loads alone was rather high which would have given uplift on the two outside piles of the group. Fortunately, it was not necessary to enlarge the cap or increase the number of piles to eliminate the uplift as the ground beams carrying the surrounding brick walls were almost on the same line as the two piles. When laid from cap to cap their weight was sufficient to counter the uplift.

Pile Caps

As the column base plates were set 12 in. below shop floor level, the pile caps were constructed in two stages, the first of which finished 14 in. below floor level to allow for 2 in. of grout. The main cap reinforcement protruded above the first stage to splice into the top 14 in. of concrete. This splice steel was placed mainly around the outside face of the cap and immediately around the column base plate to prevent

Fig. 7.—The mill bay, looking north.

movement of the second stage concrete should the holding down bolts strain under load. The H.D. bolts were cast in the first stage, being anchored at the bottom by rolled steel angles. The bolt tubes were of the precast concrete type with corrugated surfaces to ensure bond with the main concrete. The whole H.D. bolt assembly, anchorages and precast concrete tubes were set up in a template in adjacent workshops and then lifted into the shutter boxes either by hand or, where too heavy, by crane. This method ensured that the bolts were accurately set and levelled.

During concreting, pockets and reinforcement were left in the cap to carry the ground beams. After the columns had been erected, lined and levelled, grout was poured into the bolt tubes and a fine gravel concrete used to grout up the column base plate. This was drifted into the 2 in. gap from one side of the plate and vibrated through to ensure contact with the base plate. Finally, the beams were constructed and the top 14 in. of concrete laid and trowelled to give a slight fall away from the column legs.

STRUCTURAL STEELWORK

British Standards

The structural steelwork was designed in accordance with B.S.449 : 1948, together with Addendum No. 1, PD 1953, *The use of tubular members in building.*

The tubes were in Grade 15 steel complying with B.S.1775 : 1951. For outside diameters (o.d.) not exceeding $4\frac{1}{2}$ in., hot finished welded tubes were used and for larger diameters, hot finished seamless tubes.

The rolled sections and plates for girders, cleats, shoes and bases were in mild steel complying with B.S.15 : 1948.

Although site-connections between say, girders and columns, were bolted, the shop jointing was almost entirely welded. The joints in the tubular elements complied with B.S.938 : 1955, while those in conventional steelwork complied with B.S.1856 : 1952.

Layout of Structures

As will be observed from the specification in Table 1, northlight (or saw-tooth) construction was selected for all the buildings except the mill motor house (E), the roof of which was flat.

In the E.R.W. mill, finishing and inspection bays (A, B and C) it was decided to employ roof trusses 22 ft. 6 in. in span, as shown in Fig. 8. These trusses were spaced at 13 ft. 4 in. centres in the mill bay, which is 80 ft. wide, and at 14 ft. centres in the other two bays, which are 70 ft. wide. The depth of the northlight girders, between centres of top and bottom booms, was fixed at 8 ft. The northlight girders, in turn, were supported either by longitudinal girders or directly by the main columns. In many instances, the main columns were spaced longitudinally at 45 ft. centres and in one case at 90 ft. centres, which was very convenient, but the remaining columns were either at 30 ft. or 60 ft. centres, as a result of which the corresponding longitudinal girders were designed to carry the northlight girders in more than one position along their length. In Fig. 7, for example, the columns on the left are at 45 ft. centres and the longitudinal girders are loaded in the middle. The columns on the right are at 30 ft. centres, in consequence of which the longitudinal girders are loaded at any of the quarter-points, a typical example being shown in Fig. 9. Further variations of loading positions were required for 60 ft. girders, as will be seen in Fig. 13.

In the roll and machine shop and the pump and boiler house (D, F and G) use was made of very light roof trusses, 15 ft. in span. In this case the apex of the trusses was bolted to the side of the top boom of the northlight girders, which were 6 ft. $2\frac{9}{16}$ in. deep between centres of booms. The trusses were spaced

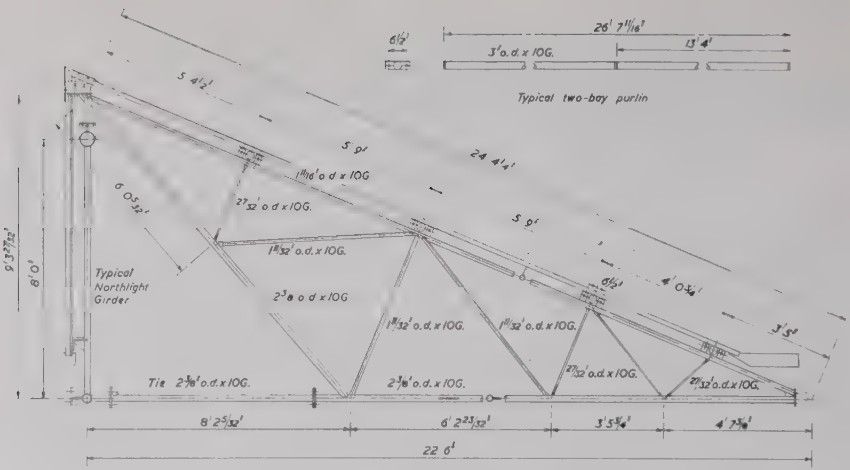

Fig. 8.—Typical northlight roof truss.

at 15 ft. centres in the roll and machine shop, which is 60 ft. wide and 13 ft. 4 in. centres in the other building, which is 40 ft. wide.

In his paper ' Joints in tubular structures ' published in THE STRUCTURAL ENGINEER for April 1959, Godfrey gave details of single-bay and two-bay tubular purlins. At Shotton, extensive use was made of a variation of two-bay purlins and side rails, typical details being shown in Fig. 8. The flanged ends of such members fit *between* the cleats on every second truss or column, as the case may be, with the result that the intermediate ' fins ' are slightly off centre. It is therefore necessary to put some paint or other distinguishing mark on the members to avoid confusion during erection. In every other respect, such members are easy to erect and efficient in use.

Cranes

Details of the various overhead cranes are given in Table 1. With the exception of those in the coil stock bay and the roll and machine shop, the cranes are provided largely for maintenance purposes.

All the E.O.T. cranes were constructed in tube, the 20 ton capacity crane for the E.R.W. mill bay being shown in Fig. 10.

Although a reduction of about 25 per cent in the dead weight of such cranes can be made by using tubes, the general layout follows normal practice, a typical half-section being shown on the left in Fig. 11. Nevertheless, the three-boom half-section shown on the right in Fig. 11, which could be readily constructed in tubes and which is more economical in steel, arouses little enthusiasm among crane manufacturers in general who show a marked reluctance in this respect to depart from long-established convention.

Crane Girders

In the three principal buildings all the 30 ft. and 45 ft. span girders were of plate girder construction, typical examples being shown in Fig. 12.

In the coil stock bay and in the roll and machine shop the 30 ft. girders comprised rolled steel joists and channels.

From the structural point of view, the 90 ft. and 60 ft. span lattice crane girders are not without interest. The 60 ft. examples shown in Fig. 13 comprise 12 in $\times$ 8 in. $\times$ 65 lb. R.S.J. top booms and $6\frac{5}{8}$ in. o.d. $\times$ $0{\cdot}375$ in. bottom booms, with web members varying from $6\frac{5}{8}$ in. o.d. $\times$ $0{\cdot}219$ in. to $4\frac{1}{2}$ in. o.d. $\times$ 5G.

For some years the tube interests in Germany and Italy have employed long crane girders of similar design, and although further examples are now under construction here, the Shotton girders are believed to be the first examples erected in the British Isles.

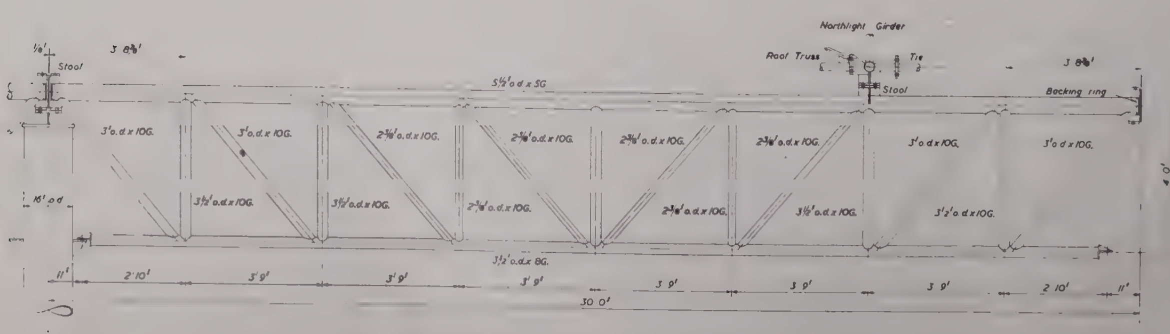

Fig. 9.—Typical longitudinal girder.

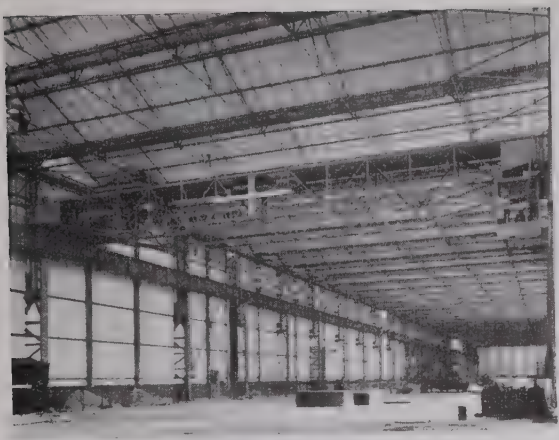

Fig. 10.—E.R.W. mill bay, looking south.

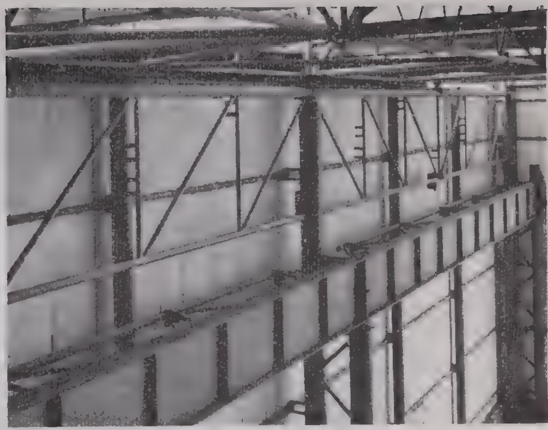

Fig. 12.—Typical crane girders.

Abroad, the top booms have usually comprised broad flange beams, but tubes and rectangular hollow sections have also been used. It would appear to be desirable, however, to have a boom with a flat top surface to facilitate the connection of the crane rails. In addition, the foreign examples have been bolted to the columns in much the same way as the other lattice girders were at Shotton. Nonetheless, at Shotton, the bottom booms of the latticed crane girders were foreshortened. Although this arrangement is sound theoretically and saves steel, there would probably be additional rigidity if the bottom boom were carried right through and attached to the columns. As visitors to the plant, albeit laymen, have already asked whether these girders are complete, there may also be aesthetic reasons for increasing the length of the booms.

The horizontal surge girders for all the crane girders were constructed in tube, typical examples being shown in Fig. 12. All the crane rails were 56 lb. per yd. bridge rails, which were bolted to the girders.

Columns

Throughout the plant, the whole of the columns were constructed in tube. Details of the most complicated of the columns are given in Fig. 5. Similar methods were employed for the other crane and roof columns, details of which are shown in Figs. 7, 12 and 13 in particular.

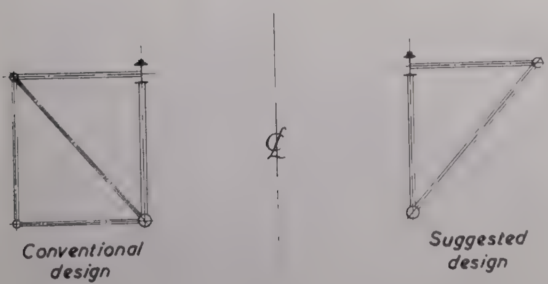

Fig. 11.—Sections of tubular E.O.T. cranes.

While there is no doubt that tubes are highly efficient as axially-loaded struts it is also true to state that rolled sections are more efficient when columns are subjected to heavy flexure. On grounds of initial cost, it would therefore be difficult to justify the use of tubes for some of the columns in the plant. There will always be a division of opinion on appearance, but from the maintenance point-of-view tubes are easy to clean and paint.

In the coil stock bay, shown in Fig. 14, the gantries comprised columns at 30 ft. centres at the north end and 45 ft. centres adjoining the roll and machine shop. These columns comprised $10\frac{3}{4}$ in. o.d. $\times$ 0·365 in. main legs and $5\frac{1}{2}$ in. o.d. $\times$ 5G. sloping legs, set 6 ft. apart at the base, with 3 in o.d. $\times$ 5G. bracings.

Motor House

In order to prevent fine sand from blowing into this building it was enclosed by 11 in. cavity brickwork and it was slightly pressurised. The roof of Briggs Bitumetal, which had a cross-fall of 1 in 40, was supported by $22\frac{1}{2}$ in. $\times$ 6 in. $\times$ 45 lb. Castella beams and 13 in. $\times$ 5 in. $\times$ 35 lb. purlins, 30 ft. in span at

Fig. 13.—West flank of inspection bay.

Fig. 14.—North section of coil stock bay.

8 ft. centres. Part of the framework can be seen on the right in Fig. 7. The columns comprised $5\frac{1}{2}$ in. o.d. × 5G. braced tubes, cruciformed at the caps to take the Castella beams and light crane girders.

Site Fabrication and Erection

All the columns and those girders not greater than 60 ft. in span were completely fabricated in the works. However, 92 northlight girders, one 90 ft. longitudinal girder and two 90 ft. crane girders were shop-fabricated approximately in halves and then welded together on the site, each girder requiring a butt-weld in each boom and two joints for one diagonal member. For this purpose, a welding table of concrete plinths and R.S.J's was provided to take two girders at a time. No one will deny that site welding is usually fairly expensive, but the spliced bolted joints, which may be used as an alternative and which are often cheaper, are not completely unobtrusive.

The bulk of the erection of the steelwork and of the work of following trades took place during the winter of 1958-59 when progress was frequently impeded by rain and by black ice forming on the steelwork.

As it was decided from the outset that it would be easier to erect from the south and the E.R.W. mill bay was the most urgent from the customer's point-of-view, erection started at the south end of that bay. The five-ton Scots derrick with 95 ft. jib chosen for the main erection was moved from south to north of this bay and then dismantled and re-erected at the south end of bays B and C. It was found possible to erect most of the steelwork in these bays from a central position. A five-ton mobile crane also on the site helped with erection inaccessible to the derrick and with off-loading. The steelwork in the small bays to the east was all erected with this mobile crane. Special equipment was brought to the site for the erection of the 90 ft. crane girders.

Erection procedure was quite simple. Columns within reach of the derrick were erected on pre-levelled packs and the longitudinal and crane girders were erected. The subsequent procedure varied during the scheme. Initially, a northlight girder was erected with one central truss attached to hold it vertical, after which the remaining trusses and purlins were filled in. Later, it was found possible to erect an almost complete northlight unit in one lift, as shown in Fig. 15.

The lines of the steelwork, square and plumb, were checked at intervals of 180 ft. and the bases grouted up to that point.

Gutter laying, sheeting and glazing followed closely behind erection.

Fig. 15.—Northlight unit being slung.

TABLE 2

Unit selling rates

Item	Quantity in tons	Weight per sq. ft. in lb.	Rate per ton	Selling price per sq. ft.
			£ s. d.	£ s. d.
Framework	654	7·0	115 10 0	7 3
Tubular crane girders	66	0·71	93 0 0	7
Crane girders	343	3·7	(A) 91 15 0 / (B) 69 15 0	2 10
Castella beams	30	0·32	93 0 0	3
H.D. materials	16·2	1·17	90 0 0	10
Door frames	7·1		81 0 0	
Gutter support	8·0		73 0 0	
Glazing	57·4		74 10 0	
Glazing walkway	11·2		77 0 0	
Crane stops	2·2		105 0 0	
Safetread	5·5		170 0 0	
	1,201	12·90		1 8
Erection (all steel items)				2
Site welding				7
Painting (on site—1 coat)				11
Ventilators				6 5
R.P.M. cladding				3
Bitumetal roofing				1 7
Glazing				3
Doors				1 3
Rainwater goods				3
Site services				
				£1 5 1

(A) Plate girders (B) Compound girders

Costs

Table 2 shows the unit selling rates based on the original tender for the steelwork, submitted over two years ago. The total area of 207,900 sq. ft. includes the coil stock bay, but excludes a canopy on the west side of the inspection bay.

FLOORS and PAVINGS

The floors throughout all the bays were composed of a 9 in. thickness of concrete, including a 1 in. granolithic finish. To prevent abrasion and dusting, the concrete was given two coats of Lithurin at least fourteen days after the concrete had been laid.

All the carriageways were constructed with a 9 in. layer of blast furnace slag, covered with a $2\frac{1}{2}$ in. tarmacadam base course and a $\frac{1}{2}$ in. carpet. The same construction was used for the tube stocking area which is 1,200 ft. long × 240 ft. wide. There are no gulleys in this area and a camber of 18 in. has been provided to ensure that all rainwater drains to the perimeter.

Book Review

An Introduction to Plasticity by William Prager. (Addison-Wesley Publishing Company, 1960, Reading, Mass., U.S.A). 9 in. × 6 in., 148 + viii pp. 72s.

This is a book of four chapters which deal with the mechanical behaviour of plastic solids, the mechanical behaviour of plastic structures, limit analysis and design, and finite plastic deformations. The title describes the book exactly, it is not an elementary sketch but will take a reader new to plasticity well into the field of present knowledge.

Professor Prager does not set out to give quick short-cut methods of dealing with complex structures but he develops the basic theory carefully so as to provide a very sound foundation on which current papers can be appraised and their value appreciated. The text deserves many hours of careful reading and a feature of each chapter is an extensive list of references.

Examples are worked out in the text and about a dozen problems are set at the end of each chapter. A single criticism is that answers to numerical problems might well have been included.

This is an ideal book for those looking for a broad text which will allow them to sharpen their wits by working at the subject instead of merely reading.

A.B.

Analysis of Braced Frames by Relaxation Method

by S. L. Lee[1], Ph.D. and R. E. Ball[2], M.S.

Synopsis

A method for analysing frames with knee braces is discussed. The analysis consists of first restraining the joints of the frame from deflecting under the action of applied loads by means of a system of fixed support forces. The latter are subsequently removed by the relaxation method. Typical expressions for the fixed support forces and the stiffness influence coefficients are given, and the procedure of analysis is illustrated by means of numerical examples.

Notation

A, B, C,	—	subscripts denoting joints
a, b	—	lengths
B_x	—	horizontal component of the reaction of the brace at joint B
D_y, E_y	—	vertical component of the reaction of the braces at joints D and E respectively
EI	—	flexural rigidity
F_B	—	fixed support force at joint B
P_{23}	—	force at point 2 due to deflection at point 3
P_{BC}	—	stiffness of joint B due to mode of deflection associated with joint C
R_B	—	residual associated with the equilibrium of forces at joint B
δ_B	—	deflection of joint B

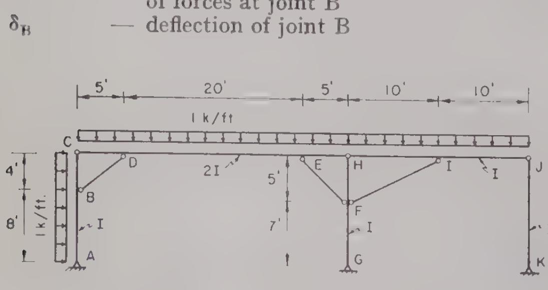

Fig. 1. (Example 1.)

Introduction

The braced frames considered in the following are those composed of columns and beams connected by knee braces. A typical frame is shown in Fig. 1, in which members AC, GH, KJ, CH, and HJ are pin-connected at joints C, H, and J, but continuous through points B, F, D, E, and I respectively. The knee braces BD, EF, and FI are two-force members pin-connected to the continuous members.

A procedure for analysing braced frames analogous to the moment distribution procedure was presented by Gerstle [1][3]. Another method of analysis using stiffness influence coefficients was discussed by Lee [2]. The method discussed in this paper is an extension of the latter.

The procedure consists of four steps. First a system of fixed support forces[4] is applied on the frame in such a manner as to restrain all the joints of the frame from deflecting under the applied loads. For the frame loaded as shown in Fig. 1, the corresponding fixed support forces are shown in Fig. 2. To remove these fixed support forces, the joints are then allowed to deflect. The three fundamental modes of deflection of the frame under consideration are shown in Fig. 3, in which the corresponding stiffness influence coefficients are also indicated. The second step, therefore, consists of determining the values of these stiffness influence coefficients. The third step involves the formulation and solution of a set of simultaneous equations which express the equilibrium conditions at the joints where the fixed support forces are applied. The solution of these equations yields the deflections of the respective joints, and the fourth step consists of calculating the forces acting in the frame with these known deflections.

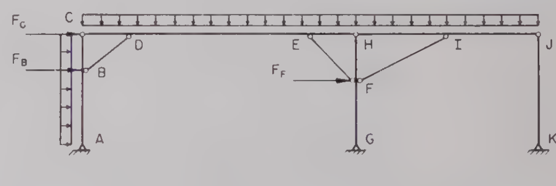

Fig. 2.—Fixed support forces. (Example 1.)

Fixed Support Forces

The number of fixed support forces necessary to restrain all the joints of a braced frame from deflecting under the applied loads is equal to the number of

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2. Teaching Assistant, Civil Engineering Department, Northwestern University, Evanston, Illinois.
3. Number in bracket refers to the listing of references at the end of the paper.
4. The term " fixed support force " is taken from [1]

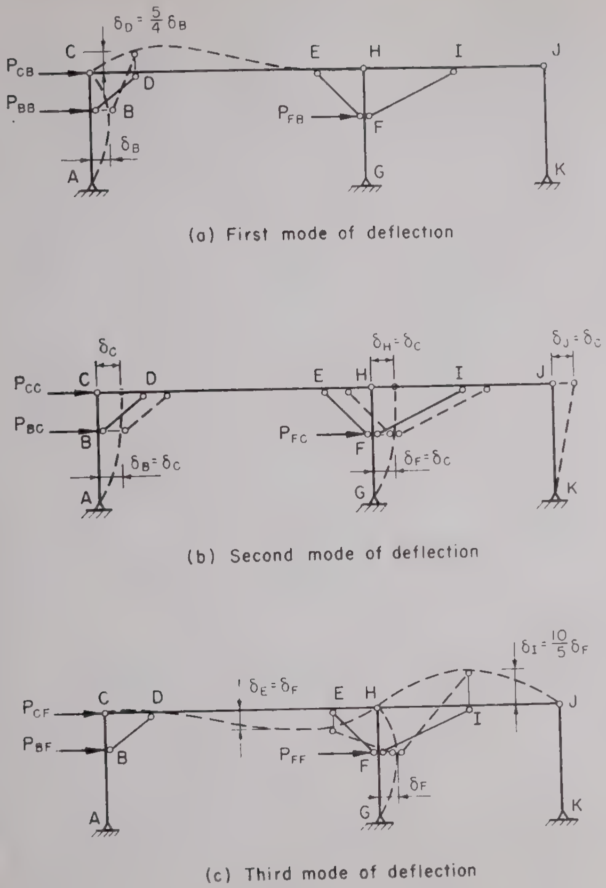
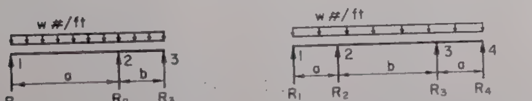


Fig. 3.—Fundamental modes of deflection. (Example 1.)

fundamental modes of deflection of the frame. For ease of calculation, the fixed support forces should be applied perpendicular to the respective continuous members, whether vertical or inclined.

Expressions for particular cases of fixed support forces for a single continuous member under uniformly distributed load, used in the following, are given in Fig. 4. For other loading conditions, the fixed support forces may be obtained from handbooks or determined by the moment distribution method, as well as other convenient procedures. In the analysis of braced frames, such as the one shown in Fig. 2, the fixed support forces F_B, F_C, and F_F are obtained readily using the expressions given in Fig. 4.



$$R_2 = \frac{w(a+b)}{2} + \frac{w(b^3+a^3)}{8ab} \qquad (1)$$

$$P_3 = \frac{wb}{2} - \frac{w(b^3+a^3)}{8b(a+b)} \qquad (2)$$

(a) Member with single brace

$$R_2 = \frac{w}{8}\left[5a+4b+\frac{b(2b^2-3a^2)}{a(2a+3b)}\right] \qquad (3)$$

(b) Member with double braces

Fig. 4.—Fixed support forces for uniformly distributed load.

Stiffness Influence Coefficients

The stiffness of a joint in a frame, for a particular mode of deflection of the frame, is the force required at the joint to produce the mode of deflection. The stiffness influence coefficient may be defined as the value of this force corresponding to unit deflection. Expressions for typical stiffness influence coefficients are given in Fig. 5. In the analysis of braced frames (Fig. 3), the stiffness influence coefficients are obtained by the combination of the coefficients shown in Fig. 5. This point is best illustrated by means of numerical examples. The first subscript in the force P (Fig. 5), denotes the joint at which P acts, and the second subscript denotes the joint where the deflection takes place.

Relaxation Method

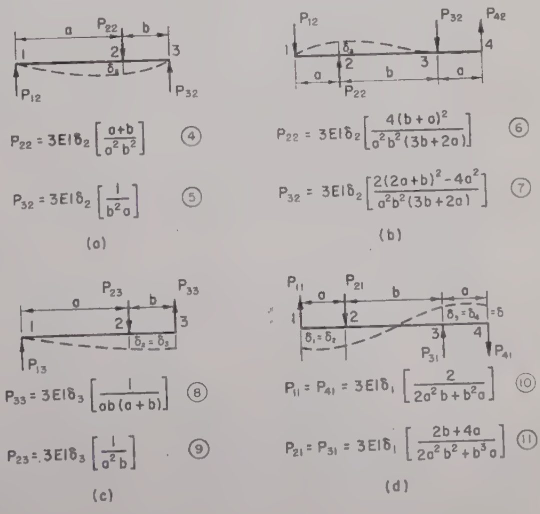
While the solution of the simultaneous equations in step 3 may be obtained by any method desired, the relaxation method [3] offers certain advantages in this analysis. It should be borne in mind that the solution of these equations yields the deflection which should take place in order to remove the fixed support forces. To this end, the relaxation method allows the successive partial removal of these forces and may be carried to any desired degree of accuracy. In the analysis of braced frames, this procedure offers the same advantage that the moment distribution method offers in the analysis of rigid frames.

In the relaxation procedure, faster convergence can be effected by means of group relaxation [4]. Physically, this is equivalent to allowing the frame to deflect in a combination of the fundamental modes of deflection. The numerical examples will demonstrate this point.

Sign Convention

Deflections

For convenience, the deflection of a joint on a vertical or inclined member is assumed positive when it is acting from left to right; and the deflection of a joint on a horizontal member is positive if it is consistent with the positive deflection of the joint on the vertical or inclined member to which it is connected by a brace.



$$P_{22} = 3EI\delta_2\left[\frac{a+b}{a^2b^2}\right] \qquad (4)$$

$$P_{32} = 3EI\delta_2\left[\frac{1}{b^2a}\right] \qquad (5)$$

(a)

$$P_{22} = 3EI\delta_2\left[\frac{4(b+a)^2}{a^2b^2(3b+2a)}\right] \qquad (6)$$

$$P_{32} = 3EI\delta_2\left[\frac{2(2a+b)^2-4a^2}{a^2b^2(3b+2a)}\right] \qquad (7)$$

(b)

$$P_{33} = 3EI\delta_3\left[\frac{1}{ab(a+b)}\right] \qquad (8)$$

$$P_{23} = 3EI\delta_3\left[\frac{1}{a^2b}\right] \qquad (9)$$

(c)

$$P_{11} = P_{41} = 3EI\delta_1\left[\frac{2}{2a^2b+b^2a}\right] \qquad (10)$$

$$P_{21} = P_{31} = 3EI\delta_1\left[\frac{2b+4a}{2a^2b^2+b^3a}\right] \qquad (11)$$

(d)

Fig. 5.—Stiffness influence coefficients.

Forces

A positive force is one which causes a positive deflection at the joint on which it acts.

Example 1.

Consider the single-storey, two-bay, braced frame loaded as shown in Fig. 1, in which the moments of inertia of the members are also indicated.

The fixed support forces F_B, F_C, and F_F (Fig. 2) will first be computed. Using (3) in Fig. 4b, the vertical fixed support force required to prevent joint D from deflecting in the vertical direction is 18·30 kips. Similarly, the horizontal fixed support force required to restrain joint B from displacing in the horizontal direction, using (1) in Fig. 4a, is —8·25 kips. The minus sign indicates that the force acts from right to left. Since both joints B and D are restrained by F_B, the latter is given by

$$F_B = (5/4)\ (18 \cdot 30) - 8 \cdot 25 = 14 \cdot 63 \text{ kips} \qquad \text{(a)}$$

the coefficient 5/4 being a function of the slope of the knee brace BD. The fixed support forces at E and I, computed in a similar manner, are respectively — 18·30 kips and 12·50 kips ; hence

$$F_F = (5/5)\ (-18 \cdot 30) + (10/5)\ (12 \cdot 50) = 6 \cdot 70 \text{ kips}$$

Since the fixed support forces at joints D, E, and I are supplied through the knee braces, the horizontal components of their reactions at these joints tend to displace the horizontal members laterally. Therefore, to restrain the frame against sidesway, the horizontal components of these fixed support forces should be held in equilibrium by F_C, which takes the value

$$F_C = (5/4)\ (-18 \cdot 30) + (5/5)\ (18 \cdot 30) + (10/5)\ (-12 \cdot 50) - 0 \cdot 50 = -30 \cdot 08 \text{ kips}$$

The fourth term, —0·50 kips, is the fixed support force required to restrain joint C in member AC from deflecting under the action of the horizontal load.

The stiffness influence coefficients for the first mode of deflection shown in Fig. 3a will now be determined. Taking $3EI\delta_B = 10^3 \delta_B k\text{-}ft^3$ for convenience, observing that $\delta_D = 5/4\,\delta_B$, and using (4) and (6) given respectively in Fig. 5a and b yield

$$P_{BB} = 11 \cdot 72\delta_B + 11 \cdot 16\delta_B = 22 \cdot 88\delta_B \text{ kips} \qquad \text{(b)}$$

The first term is the force required to cause the deformation of member AC and the second term that of member CH. The corresponding value of P_{FB} is given with the aid of (7) in Fig. 5b, or

$$P_{FB} = 6 \cdot 07\delta_B \text{ kips}$$

It should be noted that in this mode of deflection, P_{FB} only has to restrain joint E from deflecting vertically. The third stiffness influence coefficient takes the value

$$P_{CB} = -10^3\delta_B/(8)\ (4)^2 - 11 \cdot 16\delta_B - 6 \cdot 07\delta_B = -25 \cdot 04\delta_B \text{ kips}$$

The first term is given by (5) in Fig. 5a, while the second and third terms are due to the horizontal components of the reactions of the knee braces at joints D and E.

The stiffness influence coefficients for the other two modes of deflection are calculated in similar fashion. For the second mode of deflection shown in Fig. 3b, taking $3EI\delta_C = 10^3\delta_C k\text{-}ft^3$,

$$P_{BC} = 3 \cdot 91\delta_C \text{ kips}$$

$$P_{FC} = 4 \cdot 08\delta_C \text{ kips}$$

$$P_{CC} = -2 \cdot 60\delta_C - 2 \cdot 38\delta_C = -4 \cdot 98\delta_C \text{ kips} \qquad \text{(c)}$$

For the third mode of deflection shown in Fig. 3c, taking $3EI\delta_F = 10^3\delta_F k\text{-}ft^3$,

$$P_{BF} = 6 \cdot 07\delta_F \text{ kips}$$

$$P_{FF} = 9 \cdot 80\delta_F + 7 \cdot 14\delta_F + 8 \cdot 00\delta_F = 24 \cdot 94\delta_F \text{ kips}$$

Operations table				
		kips		
Row		ΔR_B	ΔR_C	ΔR_F
a	$\delta_B = 1 \cdot 00$	22·88	—25·04	6·07
b	$\delta_C = 1 \cdot 00$	3·91	— 4.98	4·08
c	$\delta_F = 1 \cdot 00$	6·07	—26·92	24·94
d	$\delta_B = 1 \cdot 00$ $\delta_C = -5 \cdot 85$	0·01	4·09	—17·80
e	$\delta_F = +1 \cdot 00$ $\delta_C = -6 \cdot 12$	—17·86	3·56	—0·03
f	$\delta_B = 1 \cdot 00$ $\delta_F = 1 \cdot 00$ $\delta_C = -7 \cdot 40$	0·02	—15·11	0·82

Table 1.—Operations table (Example 1)

$$P_{CF} = -10^3\delta_F/(7)\ (5)^2 - 6 \cdot 07\delta_F - 7 \cdot 14\delta_F - 8 \cdot 00\delta_F = -26 \cdot 92\delta_F \text{ kips}$$

To remove the fixed support forces F_B, F_C, and F_F, the amount of each mode of deflection which should take place must be such that

$$\left.\begin{aligned} R_B &= 22 \cdot 88\delta_B + 3 \cdot 91\delta_C + 6 \cdot 07\delta_F + 14 \cdot 63 = 0 \\ R_C &= -25 \cdot 04\delta_B - 4 \cdot 98\delta_C - 26 \cdot 92\delta_F - 30 \cdot 08 = 0 \\ R_F &= 6 \cdot 07\delta_B + 4 \cdot 08\delta_C + 24 \cdot 94\delta_F + 6 \cdot 70 = 0 \end{aligned}\right\} \qquad \text{(d)}$$

In other words, the values of the deflections δ_B, δ_C, and δ_F[5] in (d) should be such that the residuals R_B, R_C, and R_F are zero, or nearly so for practical purposes. The changes in the values of the residuals due to unit values of the deflections, i.e., the stiffness influence coefficients, are tabulated on rows a, b, and c in Table 1, the operations table. These three rows give the effect of each fundamental mode of deflection. For $\delta_B = \delta_C = \delta_F = 0$, the values of the residuals are equal to the fixed support forces and tabulated on row 1 of Table 2, the relaxation table. As the first trial toward reducing the values of the residuals, take $\delta_B = -1$ for which the residuals reduce to the values shown on row 2. The latter are obtained by subtracting the values on row a, Table 1 from those on row 1, Table 2. It should be noted here that there is no prominently large stiffness influence coefficient in either row a or rows b and c. Therefore, relaxing by means of the individual mode of deflection will surely result in very slow convergence. However, the speed of convergence may be increased by group relaxation, i.e., by taking different combinations of the fundamental modes of deflection such as those shown on rows d, e, and f. Using the latter, a fairly accurate solution is obtained in four operations as shown on rows 3 to 6 in Table 2. For instance, the values on row 3 are obtained by adding algebraically (—0·46) times the values on row e to those of row 2.

The total deflections are

$$\delta_B = -1 - 0 \cdot 44 + 0 \cdot 02 = -1 \cdot 42$$

$$\delta_C = (-0 \cdot 46)\ (-6 \cdot 12) + (-0 \cdot 44)\ (-7 \cdot 40) + (0 \cdot 02)\ (-5 \cdot 85) = 5 \cdot 96$$

$$\delta_F = -0 \cdot 46 - 0 \cdot 44 = -0 \cdot 90$$

5. It should be observed that only relative values of δ_B, δ_C, and δ_F are considered here since the values of $3EI\delta$ is arbitrarily taken as $10^3\delta\ k\text{-}ft^3$ for convenience. For the same reason, the stiffness influence coefficients computed are only relative values. If the correct values of the deflections are desired, however, the proper value of $3EI$ should be used.

Relaxation table				
		kips		
Row		R_B	R_C	R_F
1	$\delta_B = \delta_F = \delta_C = 0$	14·63	—30·08	6·70
2	—a	—8·25	—5·04	0·63
3	—0·46c	—0·03	—6·68	0·64
4	—0·44f	—0·04	—0·03	0·28
5	0·02d	·0·04	0·05	—0·08
6	0·01b	0·00	0·00	—0·04

Table 2.—Relaxation table (Example 1)

Since the values are relative values, no units are attached to them. With the deflections known, the forces acting in the frame are readily determined. The reactions of the knee braces will first be calculated. For instance, the horizontal component of the reaction at B is

$$B_x = (11 \cdot 72)\ (—1 \cdot 42) + (3 \cdot 91)\ (5 \cdot 96) — 8 \cdot 25 = —1 \cdot 51\ k$$

The first term in B_x is the first term of (b), the second term is given by (c) and the last term is the second term of (a). Similarly, the vertical component of the reaction at E and D are respectively

$$E_y = (6 \cdot 07)\ (—1 \cdot 42) + (7 \cdot 14)\ (—0 \cdot 90) — 18 \cdot 30 = —33 \cdot 35 \text{ kips}$$

$$D_y = (4/5)\ (1 \cdot 51) = 1 \cdot 21 \text{ kips}$$

The other reactions are calculated in a similar manner and are shown in Fig. 6. The corresponding bending moment diagram is shown in Fig. 7.

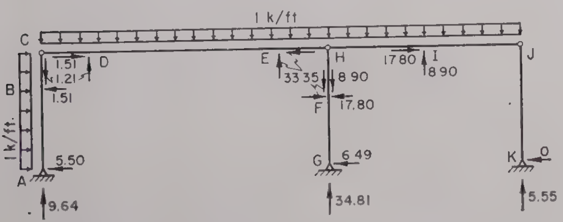


Fig. 6.—Reactions in kips. (Example 1.)

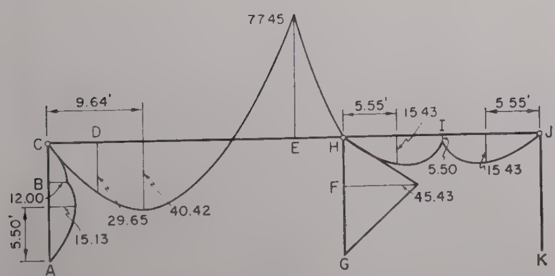


Fig. 7.—Bending moment in ft.-kips plotted on tension side. (Example 2.)

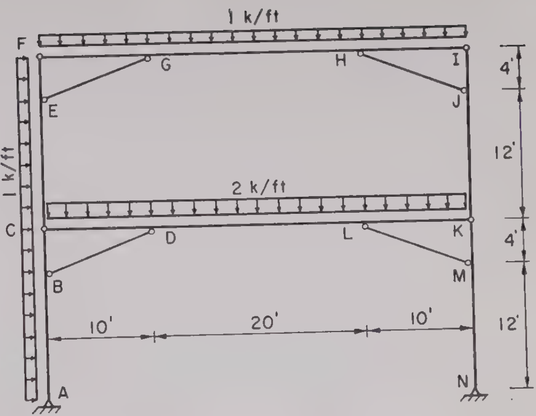


Fig. 8.—(Example 2.)

Example 2.

To illustrate the application of the procedure to the analysis of multiple storey braced frames, consider the structure loaded as shown in Fig. 8. All the members have the same moment of inertia and the columns are pin-connected at the floor levels.

The six fixed support forces are shown in Fig. 9, and the six fundamental modes of deflection in Fig. 10. The corresponding fixed support forces are tabulated on row 1 of Table 4. As in Example 1, the relative stiffness influence coefficients may be found by taking $3EI\delta = 10^3\delta\ k\text{-}ft^3$ and the coefficients thus obtained are tabulated on rows a to f in Table 3. The relaxation procedure is carried out in Table 4. The final values of the deflections are

$$\delta_B = —13 \cdot 55$$
$$\delta_M = 5 \cdot 92$$
$$\delta_C = 46 \cdot 50$$
$$\delta_E = — 5 \cdot 77$$
$$\delta_J = 3 \cdot 21$$
$$\delta_F = 17 \cdot 99$$

and the corresponding reactions and bending moment diagram are shown respectively in Figs. 11 and 12.

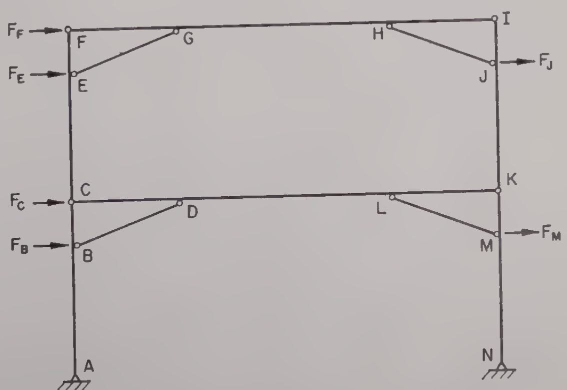


Fig. 9.—Fixed support forces. (Example 2.)

Operations table							
		kips					
Row		ΔR_B	ΔR_M	ΔR_C	ΔR_E	ΔR_J	ΔR_F
a	$\delta_B = 1$	13·98	5·47	− 17·71	0	0	0
b	$\delta_M = 1$	5·47	13·98	− 17·71	0	0	0
c	$\delta_C = 1$	1·74	1·74	− 2·60	0	0	0
d	$\delta_E = 1$	0	0	− 1·74	13·98	5·47	− 17·71
e	$\delta_J = 1$	0	0	− 1·74	5·47	13·98	− 17·71
f	$\delta_F = 1$	0	0	− 0·87	1·74	1·74	− 2·60
g	$\delta_B = 1$ $\delta_M = -1$	8·51	− 8·51	0	0	0	0
h	$\delta_E = 1$ $\delta_J = -1$	0	0	0	8·51	− 8·51	0
i	$\delta_B = 1$ $\delta_M = 1$ $\delta_C = -11{\cdot}20$	0	0	− 6·25	0	0	0
j	$\delta_B = 1$ $\delta_M = 1$ $\delta_C = -11{\cdot}20$ $\delta_E = 1$ $\delta_J = 1$ $\delta_F = -11{\cdot}20$	0	0	0	0	0	− 6·25

Table 3.—Operations table (Example 2.)

Relaxation table							
		kips					
Row		R_B	R_M	R_C	R_E	R_J	R_F
1	$\delta_B = \delta_M = \delta_C = \delta_E = \delta_J = \delta_F = 0$	76·40	89·06	− 3·33	31·86	− 44·53	1·50
2	− 9·500g	− 4·42	− 8·25	− 3·33	31·86	− 44·53	1·50
3	0·590b	− 1·19	0·00	− 13·78	31·86	− 44·53	1·50
4	− 4·700h	− 1·19	0·00	− 13·78	− 8·13	− 4·54	1·50
5	0·582d	− 1·19	0·00	− 14·79	0·00	− 1·35	− 8·80
6	0·085a	0·00	0·47	− 16·30	0·00	− 1·35	− 8·80
7	0·025b	− 0·14	0·12	− 15·86	0·00	− 1·35	− 8·80
8	0.070e	− 0.14	0.12	− 15.90	0.38	− 0.38	− 10.04
9	0·015g	− 0·01	0·00	− 15·90	0·38	− 0·38	− 10·04
10	− 0·045h	− 0·01	0·00	− 15·90	0·00	0·00	− 10·04
11	− 1·607j	− 0·01	0·00	− 15·90	0·00	0·00	0·00
12	− 2·545i	− 0·01	0·00	0·00	0·00	0·00	0·00

Table 4.—Relaxation table (Example 2)

Example 3.

The treatment of braced frames with inclined members will be illustrated by the analysis of the structure loaded as shown in Fig. 13. The fixed support forces and the fundamental modes of deflection are shown respectively in Figs. 14 and 15. It should be observed that the forces are directed perpendicular to the inclined members, and that the stiffness influence coefficients for the third mode of deflection shown in Fig. 15c are computed with the aid of (10) and (11) in Fig. 5d.

As in the previous examples, $3EI\delta$ is taken equal to $10^3 \delta$ *k-ft*^{3} for convenience. The analysis is carried out in Tables 5 and 6. The final deflections are

$$\delta_B = -0{\cdot}70$$

$$\delta_G = 0{\cdot}73$$

$$\delta_C = 0{\cdot}58$$

and the corresponding reactions and bending moment diagram are shown respectively in Figs. 16 and 17.

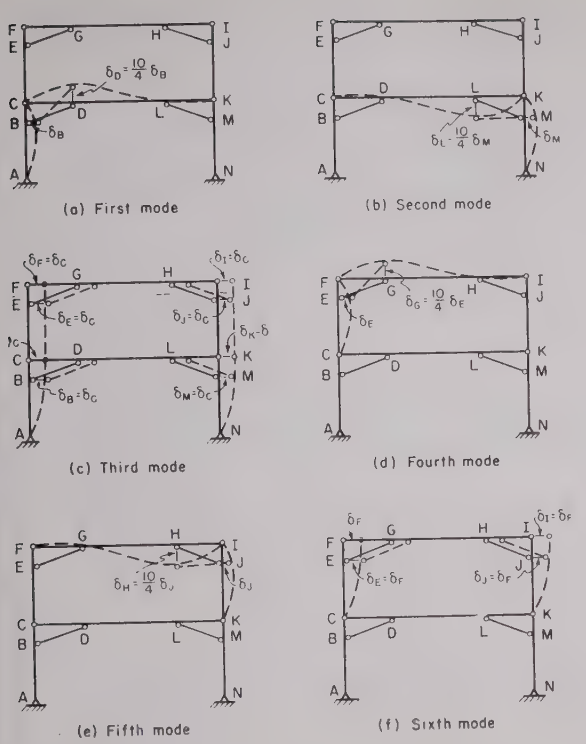


Fig. 10.—Fundamental modes of deflection. (Example 2)

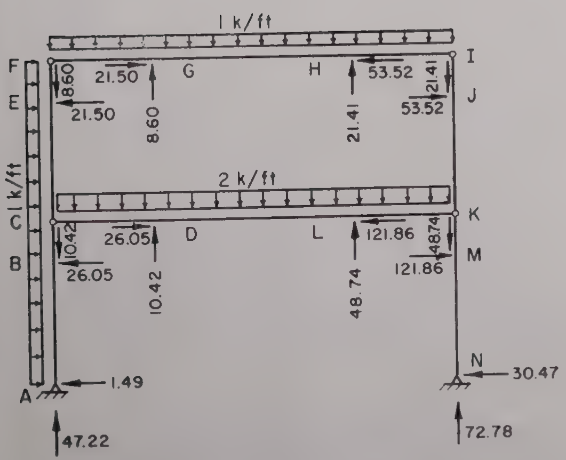


Fig. 11.—Reaction in kips. (Example 2.)

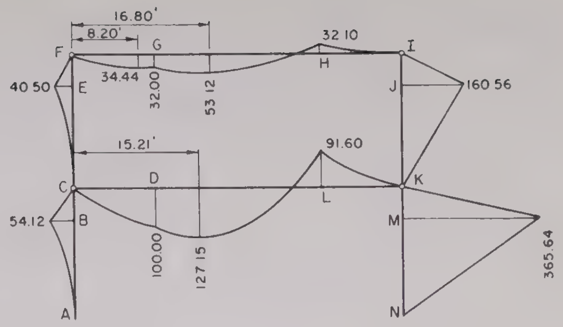


Fig. 12.—Bending moment in ft.-kips plotted on tension side. (Example 2.)

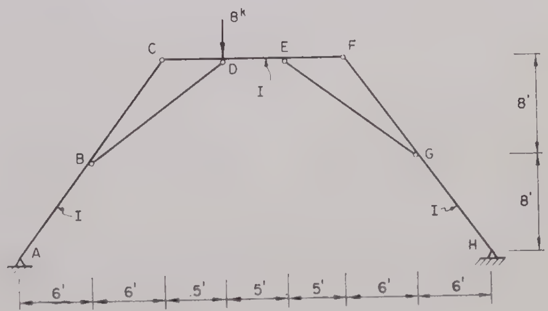


Fig. 13.—(Example 3.)

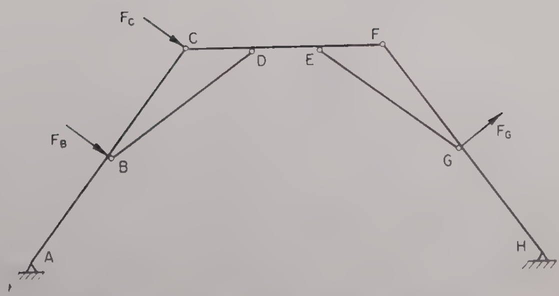


Fig. 14.—Fixed support forces. (Example 3.)

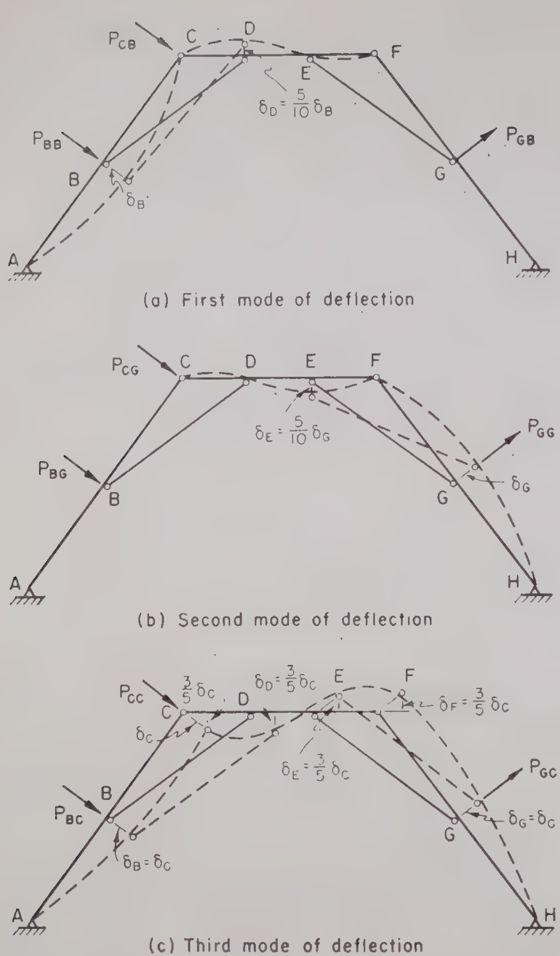


Fig. 15.—Fundamental modes of deflection. (Example 3)

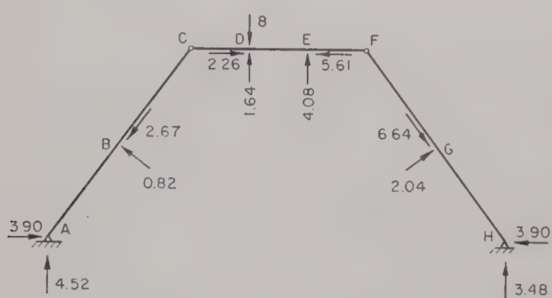


Fig. 16.—Reactions in kips. (Example 3.)

Conclusion

Although only three particular examples are illustrated, the method of analysis discussed in the foregoing may be used to advantage in dealing with many other types of braced frames. It is seen that by resorting to group relaxation, accurate results are obtained with excellent speed of convergence.

It should be reiterated here that if only the forces acting in the frames are needed, the calculation may be executed conveniently by using relative values of the stiffness influence coefficients and deflections. However, should it be desirable to have the correct values of the deflections, it is only necessary to substitute the proper values of the flexural rigidity of the members.

In the analysis of a particular frame under several loading conditions, the operations table remains unchanged. Only the fixed support forces in row 1 of the relaxation table have to be calculated for each loading condition. The relaxation process of course has to be repeated in each case.

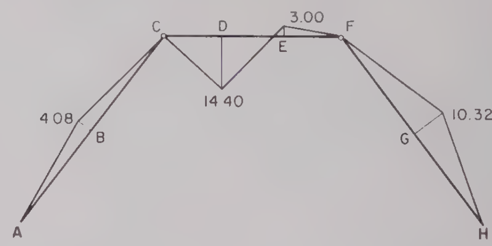


Fig. 17.—Bending moment in ft.-kips plotted on tension side. (Example 3.)

Operations table				
		kips		
Row		ΔR_B	ΔR_G	ΔR_C
a	$\delta_B = 1$	8·40	5·60	—22·61
b	$\delta_G = 1$	5·60	8·40	—22·61
c	$\delta_C = 1$	—3·81	—3·81	16·27
d	$\delta_B = 1$ $\delta_G = -1$ $\delta_C = -0{\cdot}73$	5·58	—0·02	—11·88
e	$\delta_B = 1$ $\delta_G = 1$ $\delta_C = 3{\cdot}67$	0·02	0·02	14·49
f	$\delta_B = 1$ $\delta_C = 1{\cdot}39$	3·10	0·30	0·01

Table 5.—Operations table (Example 3)

Relaxation table				
		kips		
Row		R_B	R_G	R_C
1	$\delta_B = \delta_G = \delta_C = 0$	4·00	0·00	—8·80
2	—0·70d	0·09	0·01	—0·48
3	0·03e	0·09	0·01	—0·05
4	—0·03f	0·00	0·00	—0·05

Table 6.—Relaxation table (Example 3)

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1. Gerstle, K. H., " Analysis of Braced Frames," *Proc. ASCE, Vol.* 84, No. ST2, March 1958, p. 1561.
2. Lee, S. L., " Analysis of Frames with Knee Braces," *Civil Engg., Vol.* 28, No. 9, September, 1958, p. 670,
3. Southwell, R. V., *Relaxation Methods in Engineering Science*, Oxford University Press, 1940.
4. Allen, D. N. de G., *Relaxation Methods*, McGraw-Hill, New York, 1954, p. 17.

DISCUSSION

The Council would be glad to consider the publication of correspondence in connection with the above paper. Communications on this subject intended for publication should be forwarded to reach the Institution by January 31st, 1961.

Stiffening Girders of Two Suspended Light Metal Footbridges at Alpnach, Switzerland

by K. Sutter and A. M. Mackie

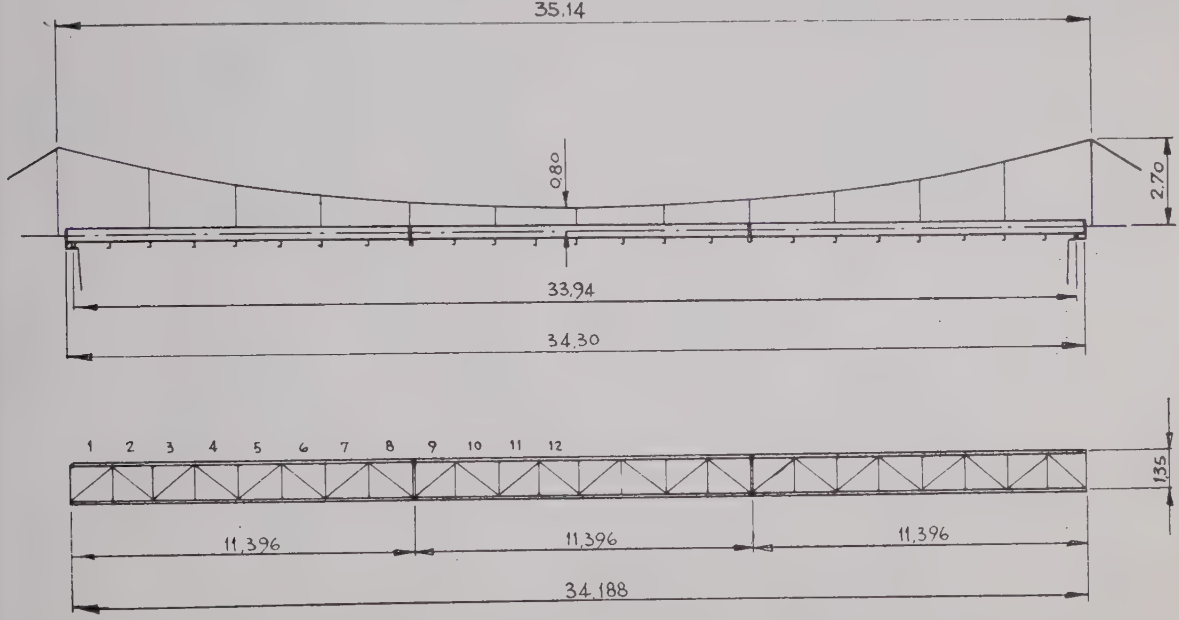

Fig. 1.—Schwand Bridge: General Arrangement

In the village of Alpnach in Switzerland there are two suspension footbridges known as the Grunder bridge and the Schwand bridge, over the Little and the Great Schlieren rivers respectively. The wooden stiffening girders of these bridges were found to rot after five or six years and in 1956 it became necessary to replace them for the third time since the bridges were built.

It was at this juncture that the firm of P. Nufer at Alpnach-Dorf proposed to the communal authorities that the new stiffening girders be made of aluminium instead of wood, the existing suspension cables being retained. Technical assistance was provided by Aluminiumwerke AG. Rorschach, and by Aluminium Laboratories Limited, Geneva, in matters of design.

1. DESCRIPTION OF THE BRIDGES

The main dimensions of the two bridges are as follows :

	Schwand Bridge	*Grunder Bridge*
Length of the stiffening girder	34·2 m.	19·6 m.
Distance between cables ...	1·35 m.	1·2 m.

The bridges are used both by pedestrians and by cattle. A general arrangement of the Schwand bridge is shown in Fig. 1.

1.1 General Arrangement

As far as possible, the same general arrangement was proposed for the stiffening girders of both bridges. The two rivers concerned are mountain streams of unpredictable behaviour, and it was deemed advisable not to reduce the cross-sectional area for the flow of water by temporary structures used in the erection of the bridges. The girders are therefore designed for erection as far as possible without falsework or mechanical lifting gear.

For this reason the stiffening girders of the Schwand bridge are divided into three sections each 11·4 m. long, and those of the Grunder bridge into two sections with lengths of 11·4 m. and 8·2 m. the 11·4 m. length being identical with the sections for the Schwand bridge.

Height of the Stiffening Girder

Taking into account the level of the suspension cables at mid-span, the appropriate camber for the stiffening girder, and the minimum length of suspender needed to provide identical suspender connections throughout, the depth of the girder is fixed at 40 cm.

Distance between Suspenders

The distance between suspenders should not exceed about ten times the girder depth, i.e. 400 cm. Since, however, it is desirable that the stiffening girder splices should correspond to the positions of the suspenders, consideration is in fact limited to only two practicable spacings :

$$(1140—12)/3 = 1128/3 = 376 \text{ cm.}$$
$$(1140—12)/4 = 1128/4 = 282 \text{ cm.}$$

(The amount of 12 cm. represents a deduction for the overhang of the stiffening girder at the bearings).

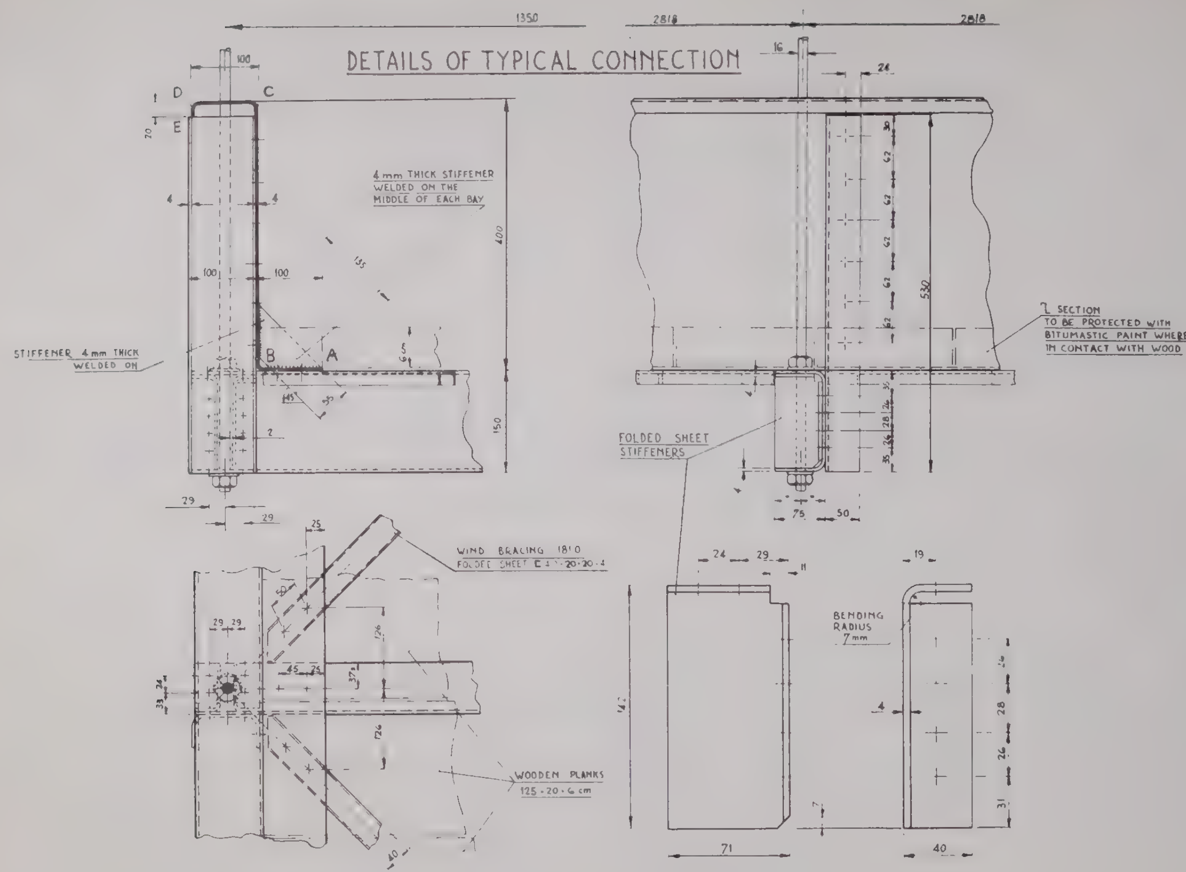

Fig. 2.—Connection between principal girder and suspenders

The shorter dimension of 282 cm. is preferred, giving a ratio between suspender spacing and girder depth of $282/40 = 7{\cdot}05$.

Position of the Deck

As the girder seating at the abutment is only about 37 cm. below the approach ramp, the deck cannot be arranged above the stiffening girder, but must be placed as low as possible between the two principal girders which make up the stiffening frame.

Type of Principal Girder

Only a plate-girder design can be considered for a girder depth of 40 cm. Since the actual stresses likely to be encountered are low (the length of a bay being only $7{\cdot}05 \times$ the girder depth), wall thicknesses can be low. It is therefore possible to form the principal girders as folded sheet sections.

Cross-Section of the Principal Girders

The type of section to be considered in this case for folded sheet is either the channel or the zed section. The zed type is adopted as providing the best compromise for simple attachment of the deck and of the suspenders.

Width of Flanges

On the basis of a web depth of 40 cm., the flange width should not exceed 10 cm., since flanges which are too wide are not fully effective over their entire width. This would give a width of 10 cm. for the deck seating on the lower flange, and if the suspenders are arranged through the centre lines of the upper flanges, the distance between the webs of the two principal girders for the Schwand bridge is 125 cm.

Thickness of the Sheet

The required sheet thickness is low, but with the above arrangement of the deck the ends of the transverse wooden boards butt against the girder webs which thus act as deep kerb. The webs should, therefore, be strong enough to support any likely impact due to the traffic load, and it was agreed that it should be not less than 4 mm. thick.

Position of the Cross-Frames

To ensure that the useful width of deck for the Schwand bridge is not less than 125 cm., the cross-frames necessary for connecting the principal girders must be arranged outside the webs and lie below the zed sections. It follows from what has been stated above that the design height of these cross-frames at the support is limited.

Cross-frames should be provided at all the points where suspenders are placed. Between each two frames provision is made for a further frame to provide the necessary additional support for the handrail.

1.2 Choice of Material

After agreement was reached on general design characteristics it was possible to specify the structural materials to be used as follows :

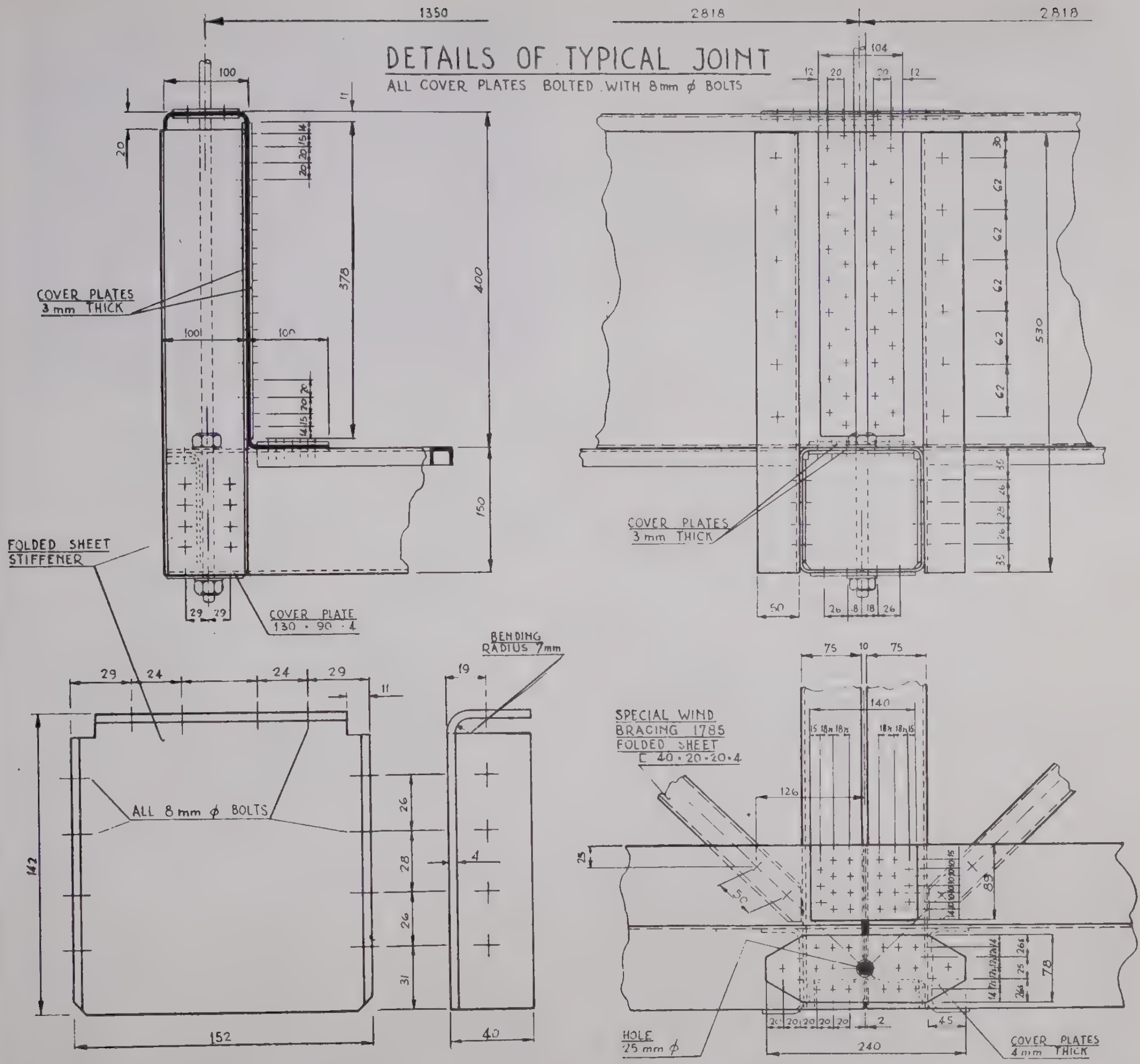

Fig. 3.—Typical joint in principal girder

Stiffening Girder

Non-heat-treated aluminium alloy sheet is adopted for the stiffening girders. The requirements are best fulfilled by the alloy 57S¹/4H equivalent to NS4¹/4H. This alloy has good corrosion and welding properties and is easy to form. Very high mechanical properties are not required.

Deck Covering

As the bridges are to be used by cattle, wooden flooring is used. The planks are placed transversely and are supported by the flanges of the principal girders. They are fastened from below by clamping clips. No holes should show on the upper surface of the planks, so as not to accelerate rotting of the timber. For the same reason a gap of 1 cm. is left between the individual planks. Impregnated larch wood is used.

Handrail

The handrail is of wood and the cross-frame members are so formed that the rail posts can be easily fixed to them and, if necessary, readily exchanged. The handrail is finally fitted only after the stiffening girder has been given the correct camber.

Methods of Joining

The 11·4 or 8·2 m. long elements of the principal girders are welded, and the site joints bolted with galvanised steel bolts. The cross-frames and subsidiary parts of the stiffening girders are joined by aluminium rivets. The rivet material should be equivalent to A56S alloy (NR6).

Suspenders

New suspenders are required since there are more than on the existing structure. They are of 16 mm. diameter galvanised steel, as also are the fastening components at the suspension cables and the stiffening girders.

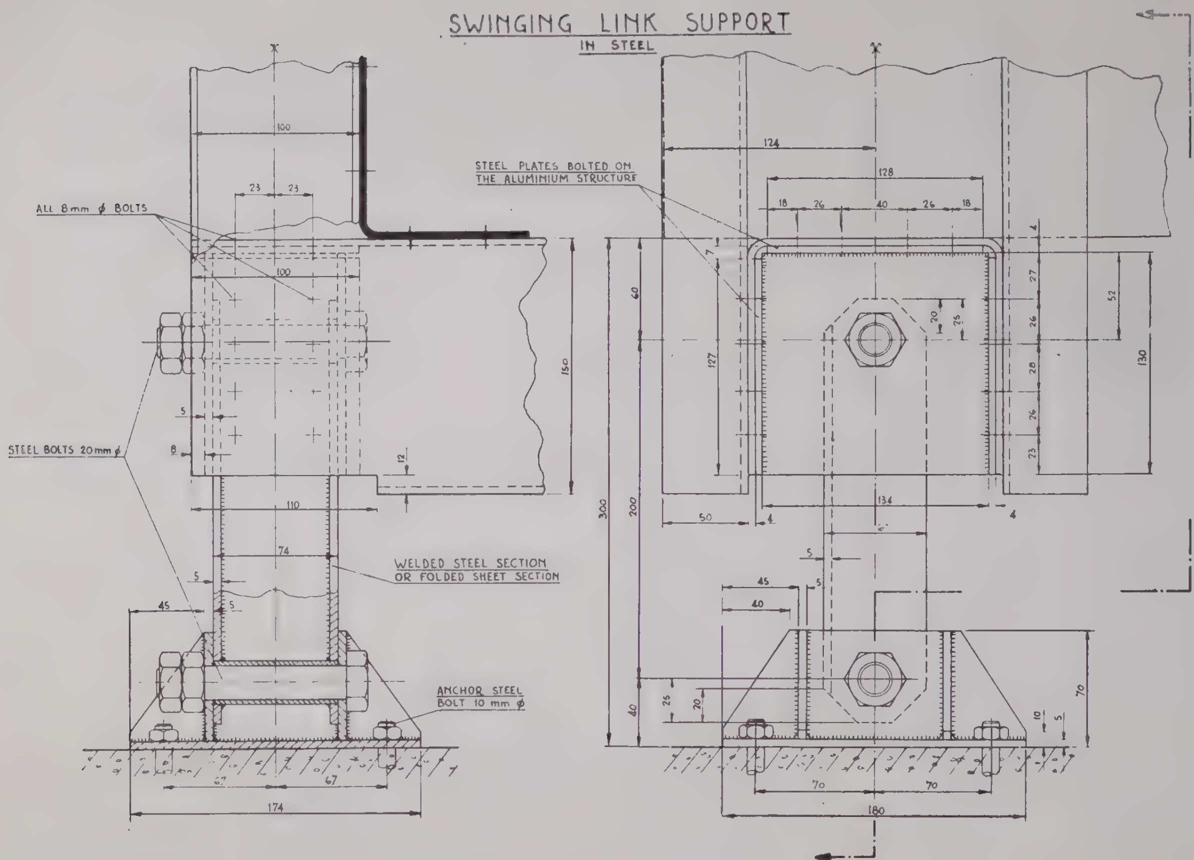

Fig. 4.—Swinging link support

Girder Supports

These are of hot-galvanised welded sheet steel. They are designed to give good service after many years even with poor maintenance.

Painting

It was decided not to paint the aluminium components except those areas of the principal girders which are in contact with the timber flooring, where bitumastic paint is used.

1.3 Structural Details

On the basis of the above specifications, design drawings were made by Aluminium Laboratories Limited, Geneva, giving enough details to invite tenders. The firm P. Nufer were general contractors for the communal authority of Alpnach and were responsible for taking down the old bridges as far as necessary, for the supply of the timber, and for the erection.

Typical Connection

Fig. 2 shows a typical connection between the girder and the suspenders. This also shows the cross-section of the principal girder (ABCDE), the position of the deck covering and of the suspenders.

In order to stiffen the lower flanges of the principal girder, welded corner stiffeners are arranged midway between the cross-frames. The upper flange is stiffened by a folded edge.

The concentrated reaction of the suspenders is transmitted to the horizontal member through two folded angle stiffeners. To simplify the construction, the vertical members of the cross-frames are connected back to back with the horizontal members or cross-beams. The eccentricities which thus arise are admissible in plate girder structures. The suspenders are threaded over a distance of 40 cm. from the lower end. This allows the cross-beam to be fixed between a nut and a lock nut and the desired camber can thus be easily obtained.

From the cross-beam the reaction of the suspender is transmitted to the vertical members which in turn are riveted to the web of the principal girder. Thus only a small reaction component is transmitted directly from the cross-beam to the principal girder.

The cross-beams pass below the principal girder and thus do not affect the deck cover. To keep the materials list to a minimum a thickness of 4 mm. was specified for all materials. This simplification was reckoned to be worth more than the few kilograms of metal involved.

The outer edge of the vertical member protrudes by 4 mm. over the upper flange of the principal girder. This prevents contact of the wooden handrail posts (Fig. 5), which are bolted to the vertical members of the cross-frames, with the principal girder.

Fig. 5.—Grunder bridge : View of the stiffening girder from below

Typical Joint

Fig. 3 shows a typical joint between the sections of the stiffening girder.

These sections are provided with a cross-frame at either end (Figs. 2 and 3), to facilitate handling and to stiffen the sections prior to assembly. In the finished structure the cross-frames are therefore doubled at the joints.

Instead of two folded angle stiffeners for transmitting the reaction from the suspenders, only an external diaphragm is provided here to stiffen the hollow section formed by the double cross-beam. A further result of the doubling of the cross-beams is that the wind bracing members are a little shorter than in normal bays.

All splices in the principal girder have double cover plates. These cover plates lie between the cross-frames with the exception of those of the upper flange.

Expansion Bearings

These are shown in Fig. 4. For a temperature variation of $\pm 30°C$, the aluminium structure alters by $\pm 0{\cdot}75$ mm. per metre. Expansion of ± 13 mm. must therefore be accommodated at each stiffening girder support. Owing to the limited design height at the supports a distance of only 27 mm. would be provided for expansion if the support were located under the cross-beams. This distance is inadequate for the required expansion, and the supports are therefore sunk into the cross-beams.

The cross-frames are doubled at the supports and the lower flanges of the cross-beams are cut away to allow the supports to be fitted. Instead of the lugs used at the other cross-members, the support itself has the function of a stiffener. Such an arrangement gives a length of 200 mm. for expansion and contraction.

1.4 Further Comments

The light metal type of bridge design was recommended to the owners for the following advantages :

(1) Long life without maintenance.
(2) Greater stability of the bridge.
(3) Higher carrying capacity.

Naturally these advantages demand higher costs. In March 1957, the Alpnach Communal Council decided in favour of the light metal design and authorised for the two bridges a total credit of Sw.Fr.20,000. The additional costs should be recovered within 15 years.

2. STATICAL CALCULATION

The statical calculation is only required to check the stresses. It is not necessary to dimension any members since these are already fixed, the supporting cables being the same as on the old structure, and the dimensions of the new stiffening girder being fixed by the design reasons given above.

This check is carried out only for the Schwand bridge since, with the aim of rationalising the construction, the same cross-sectional dimensions were used also for the Grunder bridge. The latter has a smaller span and it can be assumed to have at least the same safety factors as found for the Schwand bridge.

The Statical System

The cables are supported by steel posts at the abutments and continue beyond these points as back stays to the anchor blocks. The stiffening girder is suspended on these cables and supported on both ends by expansion bearings. The instalment of two expansion bearings is justified since at mid-span the suspenders are very short and therefore anchor the stiffening girder horizontally. The thermal expansion of the stiffening girder is thus accommodated at two points, and the movements of the ends are therefore minimised.

The calculation is divided into two parts the first being a check on the stiffening girder between two suspenders (section 2·2 below) and the second on the interaction between supporting cables and stiffening girder (section 2.4 below).

2.1 Loadings

A moving point load of 1,000 kg., representing one head of cattle, was the only prescribed load. Only this load is considered in the study of the stiffening girder between two suspenders. Compared with this the dead weight and the wind load are insignificant. On the other hand, the dead weight cannot be excluded when determining the interaction of stiffening girders and supporting cables. It is also considered desirable to determine the permissible uniformly distributed load. On the old wooden bridges, which swayed a lot, cattle had to be blindfolded and led across one at a time. With the improved stiffness and continuous deep kerb it is expected that cattle will be able to follow one another voluntarily across the new bridge. This would give in effect a uniformly distributed load of some 300 kg. per metre.

2.2 Calculation of Local Effect of Loadings

The stiffening girder consists of two folded sheet principal girders which are connected at intervals by cross-frames and horizontal bracing members. The principal girders have upper flanges stiffened by lipped edges and lower flanges which are connected together via the timber flooring.

For the calculation of the principal girders it is assumed that this connection is non-existent owing to the timber having rotted away at the fastening screws, and that therefore the two principal girders can deform independently of each other. As a result the principal girders are loaded in torsion and bending by the concentrated load. In the original article[1] of which this is an abridged version, the calculation of stresses due to torsion and bending was carried out in

TABLE 1

Load at	Cross-section at	Point of cross-section	Normal Stresses $\sigma kg/cm^2$ Due to bending	Normal Stresses Due to torsion	Normal Stresses Total	Shear Stresses $\tau kg/cm^2$ Due to bending	Due to twisting: Torsion (St. Venant)	Due to twisting: Flange bending	Shear Stresses Total	Comparative stresses $\sigma_c = \sqrt{\sigma^2 + 3\tau^2}$	Notes
$l/2$	$l/2$	A	+133·4		+133·4	0			0		
		B	+133·4		+133·4	+ 9·3			+ 9·3		
		C	—126·9	0	—126·9	+10·2	0	0	+10·2		
		D	—126·9		—126·9	+ 1·5			+ 1·5		
		E	—120·0		—120·0	0			0		
	$3l/8$	A	— 74·6		— 74·6	0			0		
		B	+179·0		+179·0	+ 4·9			+ 4·9		
		C	—183·1	0	—183·1	+ 4·1	0	0	+ 4·1		
		D	+ 65·3		+ 65·3	— 1·2			— 1·2		
		E	+ 76·1		+ 76·1	0			0		
$3l/8$	$l/2$	A	+100·0	+112·5	+212·5	0		0	0		
		B	+100·0	— 24·3	+ 75·7	+ 7·0		— 1·8	+ 5·2		
		C	— 95·1	— 17·4	—116·5	+ 7·6	0	+ 1·7	+ 9·3		
		D	— 95·1	+ 70·3	— 24·8	+ 1·1		+ 0·6	+ 1·7		
		E	— 90·0	+ 80·6	— 9·4	0		0	0		
	$3l/8$	A	— 93·3	— 56·3	—149·6	0		0	± 0·6		
		B	+224·0	+ 21·1	+236·1	+ 6·1		— 1·8	+ 4·9	+236·2	Max. tension
		C	—228·9	+ 8·7	—220·2	+ 5·1	± 0·6	+ 1·7	+ 7·4	—220·7	Max. compression
		D	+ 81·6	— 35·5	+ 46·1	— 1·5		+ 0·6	— 1·5		
		E	+101·4	— 40·2	+ 61·2	0		0	± 0·6		

great detail partly because, as shown later, the torsional loading produces stresses in excess of those due to bending and therefore could not have been ignored, and partly because it was considered useful to have a detailed example on record of what is after all a fairly complicated procedure. In the present article the need for brevity necessitated the omission of this detailed calculation and only mention of the various steps is made.

Determination of Cross Sectional Elements

The centre of gravity, principal axes and moments of inertia are found in the usual way. The shear centre torsion and warping constants are obtained by using Timoshenko's theory[2].

Calculation of Stresses Due to Torsion

This calculation is also according to Timoshenko's theory and some comments on the procedure are of interest.

Although the concentrated load of 1,000 kg. will in fact be spread over a short length of the bridge, it is nevertheless assumed to be concentrated at a point. Further, since it can be displaced only a short distance from the centre line of the bridge owing to the narrowness of the bridge and the relatively large width of the concentrated load, it is assumed that it is equally divided between the two principal girders.

The distance between the suspenders is $l = 282$ cm. Owing to the arrangement of a cross-member at $z = l/2$, the load cannot here produce a twisting moment; naturally this also applies for the positions $z = 0$ and $z = l$ where the transverse frames are fixed to the suspenders. Stressed for torsion, the principal girder has therefore a span of $l/2 = 282/2 = 141$ cm. or half the span for bending. This raises the problem of discovering the most unfavourable position of the load. This is done by calculating the stresses at several cross sections for several positions of the load and choosing, by inspection, that position giving the highest combined stress due to torsion and bending.

Calculation of Stresses Due to Bending

This is done in the normal way taking account of the fact that at $z = l/2$ the product of inertia of the principal girders is zero. Again the stresses at several cross-sections are determined for several load positions.

Combination of Stresses and Most Unfavourable Load Position

The stresses according to the calculations just mentioned are shown in Table 1. According to this table the most unfavourable load position lies at $z = 3l/8$ and not at $l/2$, as does also the cross-section with the highest stresses. Of course an intermediate load position could also be investigated, say at $z = 7l/16$, but there is no reason to expect that this would give appreciably higher maximum stresses. Since the existing stresses are relatively low in any case, this work can be omitted.

It is of interest to note from Table 1 that at certain cross-sectional points the normal stresses due to flange bending are even higher than the ordinary bending stresses; the calculation for torsion can often be, as in this case, an important one.

2.3 Calculation of the Joints

The joints in the principal girders are not dimensioned for the actual existing forces and moments, but for those which would exist were the material stressed to its proof stress. This method ensures that the rigidity of the joints is roughly equivalent to that of the principal girders. It also accounts for the large number of bolts in the joint (Fig. 6).

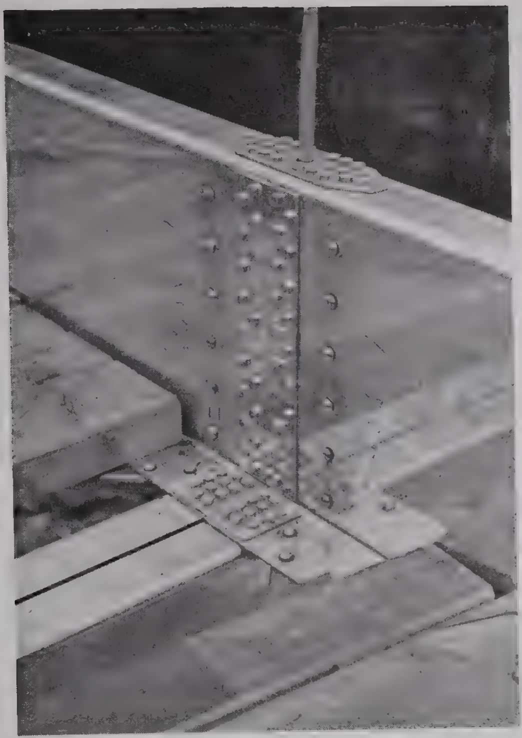

Fig. 6.—Schwand Bridge: Inside view of the stiffening girder joint

Cross Frames

The calculations for torsion and bending give the loads and moments acting on the cross frames. The stresses in the latter are calculated in the normal way while the joints are again dimensioned on a proof stress basis.

Horizontal Bracing between Girders

The horizontal bracing is calculated for load conditions which can occur during transport and erection only, its secondary function in the complete bridge as wind bracing was not checked by calculation. Adequate wind bracing is in any case provided by the wooden decking.

Deck Covering

The thickness of the planks is 6 cm. This is more than sufficient for the loads and so allows for wear and deterioration.

2.4 Calculation of Overall Effect of Loadings

The stresses and deflections due to the overall effect of the loadings are calculated according to the simple theory of suspension bridges due to Sir Alfred Pugsley which appeared in THE STRUCTURAL ENGINEER for March 1953 (3). It is therefore not necessary to detail the calculation, but of interest to recall that this theory analogizes the stiffening beam of a suspension bridge to a beam on an elastic foundation and shows that although the stiffness characteristics of the cable are different from those assumed for an elastic foundation an average value from the cable nevertheless produces a sufficiently good answer for all practical purposes. An advantage of this method is that when a readily evaluated parameter ($= \sqrt[4]{\frac{kL^4}{4EI}}$ where k is the stiffness of the foundation) exceeds the value of π the length of the beam ceases to influence the stresses and deflections of the loaded portion. the beam may therefore be considered infinitely long This greatly simplifies the formulae used and shortens the calculation.

An interesting feature of the results is that while on normal girder bridges the maximum bending stresses are produced by a load over the entire span, the maximum here occurs under a partial load placed at the end of the span as will be seen from Table 2.

Table 2 shows the summation of stresses due to the local and overall effects of the various loadings. The stress figures for a uniformly distributed load of 3·0 kg./cm. over the entire span have been omitted since they are less than those which occur under the partial loadings. The deflection under this load (which represents a file of cattle) is however greater—33·7 cm.—and is the only loading causing a net sag on the bridge.

Adjustment of Camber

The maximum stresses occur at point C (Fig. 2) on the cross section and are as follows:

Under dead load only + 520 kg./cm². tension
Under partial load at end of span — 711 kg./cm². compression

These can be equalised by adjusting the camber to 35·3 cm. The initial estimate of 30 cm. is therefore fairly close. Under this new camber the stresses become $\pm$ 616 kg./cm².

2.5 Safety Factors

The material used is 57S½H, equivalent to NS4½H with a typical 0·2 per cent proof stress as follows:

For tension 1900 kg./cm².
For compression $0{\cdot}9 \times 1{,}900 = 1{,}710$ kg./cm².

The safety factors are therefore:

For tension $1{,}900/616 = 3{\cdot}08$
For compression $1{,}710/616 = 2{\cdot}77$

When, as in this case, the calculation is carried out in great detail it is normal to employ factors of 2·25 for tension and 2·0 for compression, based on the guaranteed proof stress. The above factors are appropriately higher, being based on typical properties, and are sufficient to eliminate the possibility of fatigue failure.

2.6 Buckling

Local buckling of the lower flange of the girders is prevented by the decking while the upper flange is adequately stiffened by the lip. Check calculations were made in the normal way. Lateral buckling of the beams was also checked and found to be unimportant as a large factor of safety exists.

2.7 Cables, Suspenders and Beam Supports

The loads in the cables and suspenders and the maximum reactions at the supports are all found, from the simple theory of suspension bridges, to produce only low stresses in the material provided. This is a result of the fact that these items either existed already or were designed to their practical requirements.

TABLE 2

SUMMATION OF STRESSES DUE TO TORSION AND BENDING

Loading Condition	Maximum stresses, kg/cm^2, at point of cross-section : A	B	C	D	E	Deflection (cm)
(1) Concentrated load $P = 1{,}000$ kg.						
from 30 cm. camber	+ 220	— 530	+ 542	— 193	— 240	— 30
from P overall effect	— 249	+ 599	— 612	+ 208	+ 271	5·9
local effect	— 150	+ 236	— 220	+ 46	+ 61	0
from dead load, local effect	— 16	+ 24	— 22	+ 5	+ 6	0
Total	— 195	+ 329	— 312	+ 66	+ 98	— 24·1
(2) Partial distributed load $p = 3{\cdot}0$ kg./cm. at mid-span						
from 30 cm. camber	+ 220	— 530	+ 542	— 193	— 240	— 30
from p overall effect	— 340	+ 816	— 833	+ 301	+ 369	16·94
local effect	— 53	+ 80	— 74	+ 15	+ 19	0
from dead load, local effect	— 16	+ 24	— 22	+ 5	+ 6	0
Total	— 189	+ 390	— 387	+ 128	+ 154	— 13·0
(3) Partial distribution load $p = 3{\cdot}0$ kg./cm. at end of span						
from 30 cm. camber	+ 220	— 530	+ 542	— 193	— 240	— 30
from p overall effect	— 472	+ 1,132	— 1,157	+ 413	+ 512	12·1
local effect	— 53	+ 80	— 74	+ 15	+ 19	0
from dead load, local effect	— 16	+ 24	— 22	+ 5	+ 6	0
Total	— 321	+ 706	— 711	+ 240	+ 297	— 17·9
(4) Dead load only						
from 30 cm. camber	+ 220	— 530	+ 542	— 193	— 240	
from dead load, local effect	— 16	+ 24	— 22	+ 5	+ 6	
Total	+ 204	— 506	+ 520	— 188	— 234	

2.8 Flexural Vibrations

As a tailpiece to his elastic foundations analogy theory Pugsley shows that it can be made to produce expressions of the same form as those derived by Steinman for the flexural vibrations of suspension bridges. A simple and satisfactory check on the bridge is therefore afforded by measuring the frequency of its flexural vibrations, since these can easily be studied on the finished structure.

Using Steinman's formula the natural frequency is

$$f = A\sqrt{g/d} \cdot \sqrt{1 + B \cdot R}$$

where $g = 981$ cm./sec^2.
d = cable dip = 190 cm.
$R = 24\ EI_x d/wl^4$
w = effective dead load of bridge = 1·041 kg./cm.
E = 700,000 kg./cm^2.
I_x = 10727·4 cm^4.
l = 3394 cm.

Values of the constants A and B are given in Table 3 which gives also the resulting values of f for the first three modes of vibration.

Table 3

Mode	A	B	f	$1/f$ (sec.)
First	0·35	13·2	1·65	0·606
Second	0·49	29·6	3·225	0·310
Third	0·71	52·7	6·06	0·165

The first mode vibrations observed on the finished Schwand bridge have a frequency of 1·6 to 1·7 per second. Agreement between calculation and observation is therefore very good.

3. FABRICATION OF THE STIFFENING GIRDER

The stiffening girder was made by the firm Gebrüder Tuchschmid A.G. Frauenfeld. No account will be given of the fabrication as this was not the concern of the authors of this article, save to say that normal methods were used, that the welding was by argon tungsten arc and all the work was of high quality.

The five sections of the two stiffening girders were delivered on site in December 1957.

4. ERECTION

The method of erection of the two bridges was varied to suit local conditions. The work was observed by representatives of Aluminium Laboratories Limited, Geneva and assistance given as required.

The Schwand Bridge

The three sections forming the Schwand bridge had to be unloaded 500 m. away from the bridge site. To keep the weight of a section as low as possible (220 kg.), the decking had not yet been applied. The footpath leading to the bridge site could not be used, and the sections had to be moved across country up to the Gross Schlieren river where they were carried on a ropeway, used for moving hay, across the river and further uphill. The elements were carried by hand for the last 200 m.

The wooden superstructure was dismantled to a length of one-third from the side of the bridge where the sections arrived and the three log beams composing the 'stiffening girder' were cut by sawing at this point and removed. Transomes were fixed to the existing suspenders, some planks were laid over these transomes and the first bridge section was then placed on top of them. Since the distance between the posts which support the cables is less than 135 cm., this

Fig. 7.—Schwand Bridge : Adjustment of the camber using an auxiliary rope

section had to be carried sideways and could be placed in the normal position only after passing the posts. As soon as the first section was provisionally suspended on its own suspenders, a temporary decking was laid in position. Traffic on the bridge was interrupted for only half an hour.

A similar method was used for installing the second section. The middle third of the wooden superstructure was removed, and the second section was moved over the already installed section of the new stiffening girder and connected to the new suspenders. The procedure was repeated for the third section.

At this stage the suspension cables had assumed arbitrary positions with some suspenders taking too much and others taking no load at all. In order to correct the positions of the cables a hemp rope was hung over the posts and served as a guide to the correct catenary shape. The three sections were then bolted to each other, the height of the supports of the stiffening girder was fixed by wood blocks, and the nuts on the suspenders were tightened so that the cables and the hemp rope assumed a parallel position. The nuts were adjusted and the position of the hemp rope was corrected until the desired camber at mid-bridge was achieved approximately (Fig. 7). The bearings were then set in concrete and, to suit the prevailing temperature of 0°C, were so placed that the links were inclined by about half a centimeter towards mid-span. Final camber adjustment was then made and the decking and handrail were installed.

The bridge was opened to traffic without any further load test.

The Grunder Bridge

It was possible to unload the two sections of the Grunder bridge directly at the site. Erection was carried out as follows :

After the completion of the designs, the local authorities decided to increase the clearance of the bridge above water level by one meter. As a result, the posts supporting the cables had to be lengthened by 75 cm. and the height of the concrete ramps increased by 75 cm. This work was still in progress at the time when the installation of the bridge was started ; the cables were therefore not yet tensioned.

The old nailed plank beams were supported by four frames placed in the river bed and sawn off near the end of the span. The two sections of the new stiffening girder were placed on the supports and bolted up, and the deck covering and the handrail were fixed. Next, the suspenders were fixed and one of the slack cables was tightened with the aid of winding tackle until it showed the required sag. The end of the tension cable was then anchored and the same operation repeated for the other cable. The bridge bearings were then positioned and concreted, the final camber of the stiffening girder effected by adjusting the nuts at the suspenders, and the bridge was opened for traffic.

The critical period when the stiffening girder was supported solely by the four frames in the river bed and thus could be endangered by rising of the water level lasted under an hour.

5. CONCLUSIONS

The bridges can carry the specified load of 1,000 kg. with a generous safety factor, and have been shown to carry safely the more severe loading of a partial distributed load of 3·0 kg./cm. in its most unfavourable position. They can in fact take any string of cattle and are therefore an improvement on the old bridges. Perhaps their most noteworthy feature is their extreme simplicity of construction which was made possible by the corrosion resistant properties of the material in permitting thin sheet to be used.

REFERENCES

1. " Versteifungsträger von zwei Hängestegen in Leichtmetall" by K. Sutter and A. M. Mackie. *Technische Rundschau* No. 3 of 23rd January, 1959.
2. " Theory of bending, torsion and buckling of thin walled members of open cross-section " by S. P. Timoshenko, *Journal of the Franklin Institute*, March, April, May, 1945.
3. " A simple theory of suspension bridges " by A. G. Pugsley—*The Structural Engineer*, March, 1953, also discussion on above—*The Structural Engineer*, September, 1953.

Discussion

The Council would be glad to consider the publication of correspondence in connection with the above paper. Communications on this subject intended for publication should be forwarded to reach the Institution by January 31st, 1961.

Institution Notices and Proceedings

FORTHCOMING MEETINGS

The following meetings will be held at 11, Upper Belgrave Street, London, S.W.1.

Thursday, 10th November, 1960

Ordinary Meeting, 6 p.m. when Messrs. W. T. Brooks, T. Burnett-Stuart (Graduate), and G. B. Godfrey, A.M.I.Struct.E., A.M.I.C.E., A.M.I.Mun.E., will read a paper entitled "The 16-in. E.R.W. Tube Plant, Shotton."

Thursday, 24th November, 1960

Ordinary General Meeting at 5 p.m. for the election of members.

Thursday, 8th December, 1960

Ordinary Meeting at 6 p.m., when Mr. C. A. C. Davies, M.I.C.E., will read a paper on "The Structural Engineer in the Coal Industry."

Thursday, 15th December, 1960

Ordinary General Meeting at 5 p.m. for the election of members.

Thursday, 12th January, 1961

Ordinary Meeting at 6 p.m., when Messrs. A. Short, M.Sc., J.P., M.I.Struct.E., and W. Kinniburgh, F.R.I.C. will give a paper entitled "The Structural Use of Aerated Concrete."

Thursday, 26th January, 1961

Ordinary General Meeting for the election of members 5.55 p.m. followed by an Ordinary Meeting at 6 p.m. when Messrs. P. B. Edwards, A.M.I.Struct.E., and R. B. Rigg will read a paper on "The Design and Construction of Extensions to British European Airways Engineering Base at London Airport."

Members wishing to bring guests to the Ordinary Meetings announced above are requested to apply to the Secretary for tickets of admission.

CANCELLATION OF MEETING

It is regretted that owing to unforeseen circumstances it has been necessary to cancel the Ordinary Meeting arranged for the 15th December, 1960.

APPOINTMENT OF SECRETARY

Applications are invited for the position of Secretary of the Institution of Structural Engineers from men between the ages of 35 and 50. Applicants, who must be British, should possess a University degree, preferably with honours, or be a corporate member of a chartered engineering Institution, or have other suitable professional or service qualifications, and must have wide administrative experience.

Commencing salary will be £3,000 per annum; there is a contributory superannuation scheme.

The successful applicant will be required to take over from the retiring Secretary on the 1st October, 1961.

Further particulars may be obtained on application to The Institution of Structural Engineers, 11, Upper Belgrave Street, London, S.W.1, the envelope to be marked "Appointment of Secretary."

The closing date for the submission of the detailed applications will be 30th November, 1960.

EXAMINATIONS, JULY, 1960
HOME AND OVERSEAS CENTRES

Pass List

The Examinations of the Institution were held in July, 1960, at the usual centres in the United Kingdom and overseas at: Accra, Aligarh, Auckland, Barbados, Beirut, Bloemfontein, Bombay, Brisbane, Bulawayo, Calcutta, Cape Town, Colombo, Cooma, Dar-es-Salaam, Denver, Dunedin, Durban, East London (South Africa), Edmonton, Gwelo, Hong Kong, Jesselton, Johannesburg, Kampala, Kingston (Jamaica), Kuala Lumpur, Lahore, Lusaka, Madras, Melbourne, Montreal, Multan, Nairobi, New York, Ottawa, Rangoon, St. Louis, Salisbury (Southern Rhodesia), Singapore, Sydney, Toronto, Trinidad, Vancouver, Wellington (New Zealand), Windhoek (South Africa), Windsor (Ontario), Winnipeg, Wisconsin.

One hundred and seventy-nine candidates took the Graduateship Examination (112 at home and 67 overseas), and 817 took the Associate-Membership Examination (641 at home and 176 overseas). Of these, 63 passed the Graduateship Examination (38 at home and 25 overseas) and 118 passed the Associate-Membership Examination (97 at home and 21 overseas).

The names of the successful candidates are :—

Graduateship Examination

Agrawal, Ramesh Chandra
Ayanlaja, Thomas Omodele Modupe
Baines, Roy Taylor
Bamfield, Eric Michael
Bose, Shib Nath
Branigan, Cyril Albert
Brown, Roy Charles
Butler, Alyson Edward
Chadwick, Raymond Joseph
Chandler, Ivor Stanley
Cleworth, Graham
Dalvi, Prabhakar Laxmanrao
Davidson, Peter
Dodhia, Sobhagchand Lalji
Egan, Charles Henry Joseph Anthony
Erasmus, Jan Hendrik
Fearnley, John
Fitzpatrick, John Anthony
Gopalakrishnan, Narayanan
Gould, Charles James
Hardy, Bernard James
Hodgson, William
Johnson, James Charles
Jones, William Leslie
Kiely, Patrick Denis
Kwong Tak Cheong
Maguire, Robert
Mak Seck Hong
Markland, Frederick George
Marriott, Kenneth
Misra, Ashok Kumar
Mody, Anilkumar Ochhavlal
Nandy, Profulla Kumar
Nasim, Syed Mohammed
O'Hanlon, Terence Patrick
Oliver, Brian Vernon
Page, David George
Patel, Harikrishna Ishwarbhai
Patel, Shankarbhai Sendhabhai
Patkar, Bhalchandra Gajanan

PERINI, Flavio
PORTER, Michael John
POWELL, Malcolm Maurice
PUROHIT, Brij Kishore
QURESHI, M. Mahmood-ur-Rashid
RISHARD, Izzudheen Mohamed
ROBERTS, Charles Albert
SELKIRK, Arthur Thomas
SHENOLIKAR, Gurunath Madhav
SHETYE, Gajanan Anant
SIN TOH CHENG
SKILTON, David Alan
SMITH, Dennis Arthur
STEWART, Iain Thomas
STOCKS, Keith Bernard
STRYDOM, Charles Johannes
TAK RAM SINGH
TREZIES, John Raymond
VAZ, Jerome Edwin
VIDAL, Selwyn Francis Fitzpatrick
WATSON, Nigel Anthony
WINDSOR, Keith Charles
WONG PAK LAM

Associate-Membership Examination

ALSOP, David John Andrew
BAKER, Peter John
BARCZYNSKI, Miroslaw
BARNARD, John Desmond Nice
BARRETT, Brian Albert
BATES, Peter John
BELL, Thomas John
BIELOUS, Jaroslaw Wiktor
BIVIJI, Abbas Tayebji
BLAIN, Derek Peers
BLAKE, Ronald James
BRADLEY, John
BRINDLEY, John Hall
BROWN, Donald Robert
BURBAGE, Roy Saunt
BUTLER, Brynmor
CANNON, John Charles
CARCAS, Ronald Stanley
CHMIEL, Kazimierz Otton
CLAXTON, Brian Hugh
COOPER, Harold Kenneth
CORLETT, Derek Henry
CUFFE, Terence Neil
DAY, Thomas Michael
DINES, Edward Lawrence
DREWETT, George Iain Hay
DURRANT, Leslie Dennis
EDWARDS, Paul Harper Dickinson
EVERETT, Richard William
FAGAN, John George
FOULGER, Francis Charles Christopher
FRASER, John Scott
FUDGE, Peter Herbert
GARNER, Graham
GHAUS, Mian Ghulam
GILL, John David
GOLINSKI, Jerzy
GOLLINS, Donald Edwin
GRAY, Gordon Maxwell
GRIFFIN, Ernest John
HARRIS, Colin Frederick
HAYIM, Basil
HENFREY, Douglas
HILSON, Barry Oliver
HOCKEN, Alan
HOLDEN, Terence Lancelot
HOLLAND, Philip Garth
HOLLAND, Robert
HOUGHTON, Stanley
HOWARD, Douglas John
HURST, Bertram Lawrence
INGRAM, Leonard Victor Walter
JENKINS, Eric Graely
JENKINS, David Ernest
JOHNSON, John Richard
JOHNSON, Peter Frederick
JONES, Ian
JONES, Roy
JUDD, Henry Courtenay
KANOO, Abdul Latif
KNEE, Dawson William
LANGFORD, Joseph Kenneth
LAUROIA, Pramod Krishnan
LAWSON, Francis Noel
LEE, David John
McDONALD, John
MAKIN, Douglas William
MANTHE, Peter Eric
MARLEY, Stanley James
MATTHEWS, Douglas Thomas
MAYNARD, Arthur Leslie
MINETT, Peter Burne
MOON, Michael Roy
MUKHOPADHYAY, Amaresh
NANDI, Himotpal
NASMITH, John Struan Irving
NEWSTEAD, William Ronald
PARDOE, Frederick James
PARSONS, Arthur Robert
PATEL, Kantibhai Varadhbhai
PHILLIPS, Lionel
PLUMMER, Ivor William
POLLARD, Eustace Hughes Warren
POVEY, Anthony Thomas
PUDDEPHATT, George Ernest
REDDIN, Peter John
ROOT, Alan James
ROY, Amal Chandra
SCHALLER, Brian George Allan
SCOTT-WHITE, Raymond
SELKIRK, Arthur Thomas
SHEFFIELD, Peter
SILLETT, Donald Frank
SLACK, John Hartley
SMITH, Bryan Stafford
SOWERBY, Paul Leon
SPLILER, Robert Michael
SPRUCE, Alan Hugh
STAFFORD, Alex James
STEDMAN, Michael John
STUART, Peter Leslie
SUTTON, Noel William
TYNAN, Derek Francis
UPTON, David John
VAN BLANKENSTEIN, Louis
VAUGHAN, Colin
WALSH, Malachy
WARD, Charles Henry
WATKINS, Gerald Richard
WEBB, Ian Robin
WESTON, John
WHITE, Terence Clive
WILCOX, Clive Ernest
WILLIAMS, Barrie Charles
WILLIAMS, John Hugh Hammerton
WILTSHIRE, Dennis Edward
WINKLER, Felix Gershon
YOUNG, Richard Charles

EXAMINATIONS, JANUARY, 1961

The Examinations of the Institution will next be held in the United Kingdom and overseas on the 10th and 11th January, 1961 (Graduateship) and the 12th and 13th January, 1961 (Associate-Membership).

DRURY MEDAL AWARD—1961

The alternative subjects for this the eighth Competition, are a pipe-bridge and a petrol service station.

Graduates and Students of the Institution not over 27 years of age are invited to apply for full details to the Secretary, envelope to be marked in the top left-hand corner 'Drury Medal Award.'

The closing date for the competition is October 1st, 1961

The general conditions of the competition are as follows:

(a) The competition shall be for Graduates and Students of not more than 27 years of age.
(b) The subjects of the competition will be designs of a structural character, that is to say, involving structural design rather than planning.
(c) The subjects of design and the conditions shall be prepared and issued biennially.
(d) A Jury shall be appointed to examine the work submitted and to interview candidates if found necessary.
(e) In order to ensure that the design submitted is the unaided work of the competitor, the drawings, calculations, etc., submitted shall be endorsed by the candidate: "I declare that the work I hereby submit is my own unaided work." The declaration shall be signed by the competitor, and be either countersigned by a corporate member, or be certified as made before a Justice of the Peace, or a Commissioner for Oaths.

SESSIONAL PROGRAMME

The Literature Committee have under consideration the selection of papers for inclusion in the Sessional Programme for 1961-2. Members who may wish to offer papers during the coming Session are invited to communicate with the Secretary.

Branch Notices

LANCASHIRE AND CHESHIRE BRANCH

The following meetings have been arranged:

Tuesday, 1st November, 1960

Joint Meeting with the Institution of Civil Engineers. "Some Recent Research on Steel Framed Structures" by Mr. J. G. Nutt, B.E., Ph.D.

At the Engineers Club, Albert Square, Manchester. Commencing at 6.30 p.m. with refreshments from 5.30 p.m.

Wednesday, 9th November, 1960

Joint Meeting with the Manchester Society of Architects and the Reinforced Concrete Association.

"Multi-Storey Car Parks" by Mr. E. N. Underwood, B.Sc.(Eng.), M.I.Struct.E., M.I.C.E. (Vice-President).

Thursday, 8th December, 1960

"The Runcorn/Widnes Bridge Project," by Mr. J. K. Anderson, M.A., M.I.C.E.

Tuesday, 17th January, 1961

"Vibration Problems in Relation to Foundation design and Construction" by Mr. J. H. A. Crockett, B.Sc., A.M.I.Struct.E., A.M.I.C.E.

Except where otherwise stated the above meetings will be held at the College of Science and Technology, Manchester, commencing at 6.30 p.m. Light refreshments from 5.45 p.m.

MERSEYSIDE PANEL

The following meetings have been arranged:

Thursday, 3rd November, 1960

"Foundations with Particular Reference to Piling," by Mr. F. R. Bullen, B.Sc.(Eng.), M.I.Struct.E., M.I.C.E. (Vice-President).

At Liverpool University in the New Civil Engineering Building Brownlow Hill.

Wednesday, 30th November, 1960

"Some Unusual Aluminium Structures," by Mr. W. Hamilton, A.M.I.C.E.

At the College of Building, Clarence Street, Liverpool.

Monday, 23rd January, 1961

"Prestressed Concrete and C.P. 115," by Mr. A. J. Harris, B.Sc., M.I.C.E., M.Cons.E.

At Liverpool University in the New Civil Engineering Building, Brownlow Hill.

All meetings will commence at 6.30 p.m. and will be preceded by light refreshments from 5.30 p.m.

Joint Hon. Secretaries: Wm. S. Watts, M.I.Struct.E., A.M.I.C.E., 11, Newchurch Lane, Culcheth, Nr. Warrington, Lancs. and M. D. Woods, A.M.I.Struct.E., 8, Dennison Road, Cheadle Hulme, Cheshire.

MIDLAND COUNTIES BRANCH

The following meetings have been arranged:

Friday, 25th November, 1960

"The Architect and the Developer," by Mr. J. Seymour Harris, M.I.Struct.E., F.R.I.B.A., A.M.I.C.E., Joint meeting with the Birmingham and Five Counties Architectural Association.

Wednesday, 7th December, 1960

"Concepts of Structural Safety," by Professor R. C. Coates, Ph.D., B.Sc., M.I.C.E., Professor of Civil Engineering, Nottingham University. Joint Meeting with the East Midlands Local Association of the Institution of Civil Engineers.

At the East Midlands Electricity Board Showrooms, Irongate, Derby, at 6.15 p.m. Tea will be served from 5.45 p.m.

Friday, 27th January, 1961

"The Structural Engineer in the Field of Atomic Energy," by Mr. T. C. Waters, M.I.Struct.E. (Member of Council), Chief Structural Engineer, United Kingdom Atomic Energy Authority, Warrington. Joint Meeting with the Institute of Welding, Birmingham Branch.

Unless otherwise stated all meetings will be held at the Byng Kenrick Suite, College of Advanced Technology, Gosta Green, Birmingham, 6, at 6.30 p.m. Tea will be served from 5.45 p.m.

Hon. Secretary: S. M. Cooper, A.M.I.Struct.E., 'Applegarth,' Hyperion Road, Stourton, Nr. Stourbridge, Worcestershire.

GRADUATES' AND STUDENTS' SECTION

Friday, 4th November, 1960

"Design and Construction of the Volta Bridge," by Mr. C. R. Blackwell, of Freeman, Fox & Partners.

Wednesday, 9th November, 1960

"Precast Concrete Frame Construction," by Mr. Bryan E. Griffin, A.M.I.Struct.E., of Concrete Limited.

At the Electricity Demonstration Hall, Irongate, Derby, at 6.15 p.m. preceded by tea at 5.30 p.m.

Friday, 2nd December, 1960

"Structural Engineering in the U.S.A.," by Mr. J. W. Fortey, B.Sc., M.S., A.I.Mun.E., A.M.I.Struct.E., recently lecturer at the University of Texas. This meeting will be of general interest, and will be open to wives and friends.

Friday, 6th January, 1961

"Precast Concrete Frame Construction," by Mr. Bryan E. Griffin, A.M.I.Struct.E., of Concrete Limited.

Friday, 20th January, 1961

Fourth Annual Buffet Dance, at the Station Hotel, Dudley.

Unless otherwise stated, all meetings will be held at the Engineering Centre, Stephenson Place, Birmingham, and will start at 6.30 p.m. Tea will be served from 6 p.m.

Hon. Secretary: H. T. Dodd, Shepherd's Cottage, Grove Lane, Wishaw, Sutton Coldfield, Warwickshire.

NORTHERN COUNTIES BRANCH

The following meetings have been arranged :—

Tuesday, 1st November, 1960

At Middlesbrough. "Lightweight Fire Protection and the Structural Engineer" by Mr. A. R. Mackay, A.M.I.Struct.E.

Wednesday, 2nd November, 1960

The above paper will be repeated at the Neville Hall, Newcastle.

Tuesday, 6th December, 1960

At the City Hall, Newcastle, 7 p.m. Joint Management Committee Meeting. "Man and Moloch" by Mr. J. R. T. Gibson Jarvie, of the United Dominions Trust.

Tuesday, 6th December, 1960

At Middlesbrough. "Clifton Bridge, Nottingham" by Messrs. J. C. Adamson, B.Sc., A.M.I.C.E., A.M.I.Mun.E., and A. Goldstein, B.Sc.(Eng.), A.M.I.Struct.E., A.M.I.C.E.

Wednesday, 7th December, 1960

The above paper will be repeated at the Neville Hall, Newcastle.

Tuesday, 3rd January, 1961

At Middlesbrough. Students' and Graduates' Meeting. "Design of Welded Plate Girders" by Mr. O. A. Kerensky, B.Sc., M.I.Struct.E., M.I.C.E.

Wednesday, 4th January, 1961

At Newcastle. Joint Meeting with the Northern Architectural Association. "Value for Money in Building" by Mr. E. S. Benson, M.B.E., M.A., F.R.I.B.A.

Unless otherwise stated, meetings in Newcastle will be held at the Neville Hall and those in Middlesbrough at the Cleveland Scientific and Technical Institution, and will commence at 6.30 p.m., preceded by a buffet tea at 6 p.m.

Hon. Secretary: P. D. Newton, B.Sc.(Eng.), M.I.Struct.E., A.M.I.C.E., Messrs. S. Cussons and Partners, 112, Borough Road, Middlesbrough.

NORTHERN IRELAND BRANCH

The following meetings have been arranged :-

Tuesday, 1st, November, 1960

"Design and Construction of a Factory," by Mr. Harold Cunningham, B.Sc., A.M.I.C.E.

Tuesday, 6th December, 1960

"Lift Slab Construction" by Mr. F. R. Benson. Joint Meeting with the Northern Ireland Association of the Institution of Civil Engineers.

Tuesday, 3rd January, 1961

To be announced.

Meetings will be held at the Civil Engineering Department, David Keir Building, Queen's University, Belfast at 6.30 p.m. unless otherwise notified. Tea will be served from 5.45 to 6.30 p.m.

Hon. Secretary: L. Clements, A.M.I.Struct.E., A.M.I.C.E., A.M.I.Mun.E., 3, Kingswood Park, Cherryvalley, Belfast, 5.

SCOTTISH BRANCH

Tuesday, 8th November, 1960

Meeting at Edinburgh.—See Edinburgh Section.

Tuesday, 15th November, 1960

Joint Meeting with the West of Scotland Branch of the Institute of Welding. "Universal Beams—After Two Years," by Mr. A. W. Turner.

Wednesday, 7th December, 1960

Meeting at Edinburgh.—See Edinburgh Section.

Tuesday, 13th December, 1960

"The Use of Timber as a Structural Medium," by J. H. Jaap, A.M.I.Struct.E.

Tuesday, 24th January, 1961

"The Provisions of the Revised British Standard for the Use of Structural Steel in Building"—B.S.449/1959, by Mr. Walter C. Andrews, O.B.E., M.I.Struct.E., M.I.C.E. (Past President).

Wednesday, 25th January, 1961

Meeting at Edinburgh.—See Edinburgh Section.

Unless otherwise stated, meetings will be held at the Institution of Engineers and Shipbuilders, 39, Elmbank Crescent, Glasgow, commencing at 7 p.m.

EDINBURGH SECTION

Tuesday, 8th November, 1960

Joint Meeting with the Edinburgh Architectural Association. "The Relationship Between Architect and Structural Engineer," by Mr. W. Underwood, F.R.I.B.A., at the Adelphi Hotel, Coburn Street, Edinburgh, at 6.15 p.m.

Wednesday, 7th December, 1960

Joint Meeting with the Edinburgh and East of Scotland Association of the Institution of Civil Engineers. "Zambesi Hydro Electric Development at Kariba—First Stage" by Mr. T. A. L. Paton, C.M.G., B.Sc., M.I.Struct.E., M.I.C.E., and Mr. C. I. Blackburn.

At the North British Hotel, Edinburgh, 6 p.m.

Wednesday, 25th January, 1961

"The Provisions of the Revised British Standard for the Use of Structural Steel in Building—B.S.449/1959" by Mr. Walter C. Andrews, O.B.E., M.I.Struct.E., M.I.C.E. (Past President).

At the Heriot-Watt College, Edinburgh, 6.30 p.m.

Hon. Secretary: W. Shearer Smith, M.I.Struct.E., A.M.I.C.E., c/o The Royal College of Science and Technology, George Street, Glasgow, C.1.

SOUTHERN BRANCH

Friday, 4th November, 1960

The Inaugural meeting of the Branch will be held at Southampton. The Chairman of the new Branch is Mr. J. J. Leeming, B.Sc., M.I.Struct.E., M.I.C.E.

Hon. Secretary: A. P. K. Tate, B.Sc., A.M.I.Struct.E., Department of Civil Engineering, the University, Southampton.

SOUTH-WESTERN COUNTIES SECTION

Hon. Secretary: C. J. Woodrow, J.P., "Elstow," Hartley Park Villas, Mannamead, Plymouth, Devon.

WALES AND MONMOUTHSHIRE BRANCH

The following meetings have been arranged :

Wednesday, 2nd November, 1960

Chairman's Address, by D. Manolopoulos, M.I.Struct.E., at Swansea.

Thursday, 10th November, 1960

Joint Meeting with the Institution of Civil Engineers. "Engineering Education in the U.S.A.," by Professor B. Neal, M.A., Ph.D., A.M.I.C.E. at Cardiff.

Tuesday, 22nd November, 1960

The Chairman's Address will be repeated at Cardiff.

Tuesday, 6th December, 1960

Joint Meeting with the South Wales Institute of Architects, Western Branch. "Brazil, Venezuela and Mexico," by Mr. Alex J. Gordon, Dip.Arch., A.R.I.B.A. at Swansea.

Meetings will commence at 6.30 p m. Those at Cardiff will be held at the South Wales Institute of Engineers and those at Swansea at the Mackworth Hotel.

Hon. Secretary : W. D. Hollyman, A.M.I.Struct.E., 41, Greenfield Avenue, Dinas Powis, Glamorgan.

WESTERN COUNTIES BRANCH

The following meetings have been arranged :

Thursday, 17th November, 1960

Mr. F. C. Greenfield, A.Am.Soc.E., A.M.I.C.E., A.M.I.W.E., will present a paper entitled "The Trend in Engineering Works of Steel, Reinforced and Prestressed Concrete—Some Continental Impressions."

Friday, 2nd December, 1960

"The Characteristics of Friction Grip Bolt Joints," by Dr. M. S. G. Cullimore.

All meetings will be held in the small lecture theatre of the University Engineering Laboratories, University Walk, Bristol, 8, at 6 p.m., preceded by tea at 5.30 p.m.

Friday, 6th January, 1961

Joint Meeting with the Reinforced Concrete Association. "Lift-Slab Design and Construction," by Mr. F. R. Benson, B.Sc.(Eng.), A.M.I.C.E.

Hon. Secretary : A. C. Hughes, M.Eng., A.M.I.Struct.E., A.M.I.C.E., 21, Great Brockeridge, Bristol, 9.

THE YORKSHIRE BRANCH

The following meetings have been arranged :

Wednesday, 16th November, 1960

At Leeds. Joint meeting with the West Yorkshire Society of Architects and the Yorkshire and Lincolnshire Branch of the Institution of Highway Engineers. "Multi-Storey Car Parks," by Mr. E. N. Underwood, B.Sc.(Eng.), M.I.Struct.E., M.I.C.E. (Vice-President).

Wednesday, 7th December, 1960

At Sheffield. "Recent Prestressed Concrete Work in North America and the Commonwealth," by Mr. Donovan H. Lee, B.Sc.(Eng.), M.I.Struct.E., M.I.C.E., M.I.Mech.E. (Hon. Curator).

Wednesday, 14th December, 1960

At Leeds. "The Use of the Electronic Computer in Structural Engineering," by Dr. E. Lightfoot, M.I.Struct.E., A.M.I.C.E.

Wednesday, 18th January, 1961

At Leeds. "Research for the Concrete Industry," by Dr. A. R. Collins, M.B.E., M.I.Struct.E., M.I.C.E. (Member of Council).

Meetings at Leeds will be held at the Metropole Hotel, King Street, and those in Sheffield at the Royal Victoria Hotel, unless otherwise stated. Meetings will commence at 6.30 p.m., preceded by a buffet tea at 6.15 p.m.

Hon. Secretary : W. B. Stock, A.M.I.Struct.E., 34, Hobart Road, Dewsbury, Yorks.

UNION OF SOUTH AFRICA BRANCH

Hon. Secretary : E. B. Kretzchmar, A.M.I.Struct.E., P.O. Box 3306, Johannesburg, South Africa.

Natal Section Hon. Secretary : J. C. Panton, A.M.I.Struct.E., A.M.I.C.E., c/o Dorman Long (Africa) Ltd., P.O. Box 932, Durban.

Cape Section Hon. Secretary : R. F. Norris, A.M.I.Struct.E., African Guarantee Building, 8, St. George's Street, Cape Town.

EAST AFRICAN SECTION

Hon. Secretary : K. C. Davey, A.M.I.Struct.E., P.O. Box 30079, Nairobi, Kenya.

SINGAPORE AND FEDERATION OF MALAYA SECTION

Hon. Secretary : W. N. Cursiter, B.Sc., A.M.I.Struct.E., A.M.I.C.E., c/o Redpath, Brown & Co. Ltd., P.O. Box 648, Singapore.

NIGERIAN SECTION

Hon. Secretary : A. Brimer, A.M.I.Struct.E., Brimer, Andrews and Nachshen, Private Bag Mail 2295, Lagos, Nigeria.

ADDITIONS TO THE LIBRARY

American Welding Society Welding Handbook, 4th Edition, Section III, Special Processes and Cutting. New York and London, 1960.

British Standards Institution Annual Report, 1959-1960. London, 1960.

Building Research, 1959. Report of the Building Research Board with the Report of the Director of Building Research. London, 1960.

The Directory of Opportunities for School Leavers. London, 1960.

FRANKLIN, P. *Differential Equations for Engineers*, 2nd Edition, New York, 1960.

GRINTER, L. E. *Design of Modern Steel Structures*, 2nd Edition. New York, 1960.

Hydraulics Research, 1959. Report of the Hydraulics Research Board with the Report of the Director of Hydraulics Research. London, 1960.

International Association for Bridge and Structural Engineering, Preliminary Publication of the 6th Congress, Stockholm, 1960, Zurich, 1960.

JAEGER, T. *Technischer Strahlenschutz I. Strahlenabschirmung durch Beton.* II Beseitigung radioaktiver Abfallstoffe, Munich, 1959.

KENT, L. E. and LOVEJOY, E. G. *Examples of Structural Steel Design to Conform with the Requirements of British Standard* 449 1959. London 1960. Presented by Mr. L. E. Kent.

PFLÜGER Alf. *Elementare Schalenstatik*, 3rd Edition. Berlin/Göttingen/Heidelberg, 1960.

PRAGER W. *An Introduction to Plasticity*. Massachusetts U.S.A. 1959.

RUNCORN, S. K., /*Editor*, *Methods and Techniques in Geophysics*, Vol. I. New York and London, 1960.

SANSOM, R. C. *Organization of Building Sites*. London, 1959.

SWAINGER, K. *The Analysis of Deformation*, Vol. IV. Waves and Vibrations. London, 1959, Presented by Mr. F. D. C. Henry.

Symposium on Nuclear Reactor Containment Buildings and Pressure Vessels, Royal College of Science and Technology. Glasgow, 1960. Preprints of Papers.

ERRATA

THE STRUCTURAL ENGINEER, August, 1960. *Some Points of Structural Interest at Calder Hall 'A' Nuclear Power Station*, by W. S. Watts.

Page 260, col. 1, paragraph 4
For 6 : 3 : 1 *read* 6.3 : 1

Page 264, col. 1, paragraph 1
For 2,400 cu. yds. *read* 7,400 cu. yds.

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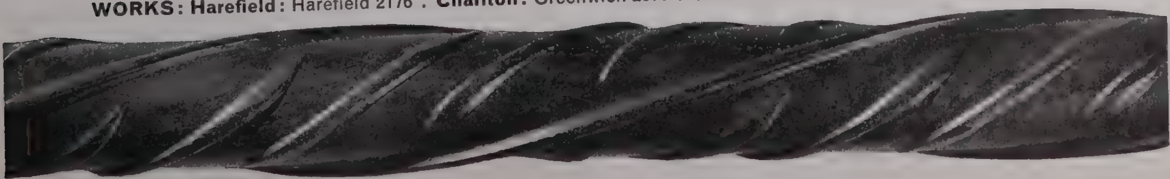

OFFICIAL APPOINTMENTS

AIR MINISTRY Works Designs Branch requires in London Structural Engineering Designer Draughtsmen for reinforced concrete or structural steelwork of all types. Applicants must have adequate training and several years' experience. Some site experience and possession of recognised technical qualification an advantage. Financial assistance and time off may be allowed for recognised courses of study. Promotion and pension prospects. Five day week with 18 days paid leave per year initially. Salary ranges from £805 (at age 25) to £980 p.a. Commencing salary dependent on age, qualifications and experience. Applicants, who must be natural born British subjects, should write (quoting Order No. Kings Cross 3745) to Air Ministry W.G.d, Lacon House, Theobalds Road, London, W.C.1, or to any Employment Exchange giving age, details of training, qualifications, full particulars of former posts held and copies of any testimonials. Candidates selected will normally be interviewed in London and certain expenses reimbursed.

ASSISTANT ENGINEER (CIVIL)

Required for their London Office by the CROWN AGENTS FOR OVERSEA GOVERNMENTS AND ADMINISTRATIONS for appointment to pensionable establishment on probation for two years. Commencing salary between £595 per annum at age 21, £830 at age 25, and £1,125 at age 34 or over, in scale rising to £1,300. Prospects of promotion. Fully qualified officers at least 27 years of age may be eligible for special increase of £75 after two years' service. Liberal leave. Five day week. Candidates must be Corporate members of the Institution of Civil Engineers or the Institution of Structural Engineers or, if below the age of 28, must have passed Parts 1 and 2 of the Associate Membership examination of the Institution of Civil Engineers or hold an exempting degree, or have passed the Associate Membership examination of the Institution of Structural Engineers. They should have had experience in the design of bridges or other structures in steel or reinforced concrete. Site experience an advantage. Candidates must be prepared to spend periods on site surveys overseas in which event special overseas allowances are payable.

Apply to CROWN AGENTS, 4, Millbank, London, S.W.1, for application form and further particulars, stating age, name, brief details of qualifications and experience and quoting reference M2A/42637/SAD.

BOROUGH OF SUTTON AND CHEAM. Borough Engineer and Surveyor's Department. Applications are invited from qualified persons for the appointment of an Assistant Structural Engineer, Grade APT V (£1,310 to £1,480 per annum) plus the appropriate London "Weighting" £45 per annum, to be engaged on the design of steel and reinforced concrete structures comprising industrial buildings, pumping and generating stations, multi-storey flats, etc., and be able to handle contracts; to supervise the checking of designs of framed structures submitted under the Building Byelaws. The Council may consider the provision of housing accommodation where necessary. The appointment, which is terminable by one month's notice in writing on either side, is on the permanent staff of the Corporation, subject to the provisions of the Local Government Superannuation Acts, 1937/53 and to the National Scheme of Conditions of Service. The successful candidate will be required to pass a medical examination. Form of application may be obtained from Mr. C. Needham, M.I.C.E., M.I.Mun.E., A.M.I.Struct.E., Borough Engineer and Surveyor, to whom it should be returned accompanied by copies of two recent testimonials, not later than 18th November, 1960, endorsed "Assistant Structural Engineer." Applicants must state whether they are related to any member or holder of any senior office under the Borough Council. Canvassing in any form will disqualify.

STRUCTURAL ENGINEERS

FOR THE ARCHITECTS DEPARTMENT OF THE LONDON COUNTY COUNCIL

Applications are invited for engineering assistants (up to £950), Engineers Grade III (£830—£1250) and Engineers Grade II (£1250—£1500) in the Structural Engineering Division and the District Surveyors' Service.

Structural engineers are employed:

IN THE STRUCTURAL ENGINEERING DIVISION AT THE COUNTY HALL

1 as designers. The department has a programme of new multi-storey buildings of many types.

or 2 examining structural schemes submitted by developers. Many schemes go to the bounds of existing knowledge and provide a fascinating variety of work at the top level of professional quality. The Structural Engineer has to advise the Council on whether schemes should be approved or not.

or 3 on research and development. New materials and new forms of construction are constantly being investigated and a close liaison is maintained with other divisions of the department to advise on the engineering aspects of their development work.

IN THE DISTRICT SURVEYORS' SERVICE

There are 28 District Surveyors in London, generally one for each borough. Each District Surveyor has a small staff of engineers according to the size of the district. There are vacancies for structural engineers who would have an excellent opportunity to study for the District Surveyors' examination with a view to competing for appointment as District Surveyors when vacancies occur. Assistants have to take considerable responsibility and their work is out of doors.

There are opportunities for promotion on merit to the following grades:

STRUCTURAL ENGINEERING DIVISION

Senior Structural Engineer	£2,400—£2,700
Assistant Senior Engineer	£1,750—£2,050
Principal Assistant (Professional)	£1,700—£1,950
Grade I	£1,500—£1,700

DISTRICT SURVEYORS' SERVICE

(provided that the qualifying examination has been passed).

District Surveyor	Range from £1,850—£3,700
Assistant District Surveyors	£1,500—£1,700

(plus an allowance of £61 10s.)

Within all grades the scales are flexible and individuals whose performance merits rapid advancement can be placed on higher points in the scale in addition to receiving normal annual increments. All positions are pensionable and officers become permanent after two years satisfactory service.

Further particulars and application forms from

HUBERT BENNETT, F.R.I.B.A., Architect to the Council County Hall, S.E.1, quoting ref. EK/SE/2315/11.

OFFICIAL APPOINTMENTS—continued

COUNTY BOROUGH OF HUDDERSFIELD.—Borough Engineer and Surveyor's Department. Applications are invited for the appointment of a Bridges Assistant at a salary in accordance with Scale A.P. and T.V. (£1,310—£1,480). Candidates should have considerable experience in bridge design and construction and preferably be corporate or associate members of the Institution of Structural Engineers. The post is superannuable and subject to satisfactory medical examination. Applications, stating age, qualifications and experience, together with the names of two persons to whom reference can be made, should reach A. L. Percy, Esq., M.I.C.E., Borough Engineer and Surveyor, High Street Buildings, Huddersfield, not later than 8th November, 1960. Canvassing is prohibited and will be considered a disqualification.

SURREY COUNTY COUNCIL.—Applications invited for the appointment of Structural Engineers on Grade Special—IV (£840 to £1,310 p.a. plus £40/45 p.a. London Allowance). Must be qualified Structural or Civil Engineer and have had experience in design and detailing of steelwork and/or reinforced concrete for medium to large scale contracts. Approved removal expenses will be paid to successful candidates in this Grade. Candidates will be appointed at the appropriate point within the scale according to age and ability. Full details, present salary and three copy testimonials to County Architect, County Hall, Kingston, as soon as possible.

WESTERN REGION OF BRITISH RAILWAYS requires Engineering Assistants (capable of supervising Drawing Office staff), experienced in the design of reinforced concrete bridges and other structures to fill vacancies in posts in the salary range £1,095/1,150. Interesting work in pleasant conditions with promotion on Merit; superannuation fund; reduced rates of travel and other concessions; five day week. Applications, giving age, qualifications and experience to, Chief Civil Engineer, British Railways, Western Region, Paddington, London, W.2.

WESTERN REGION OF BRITISH RAILWAYS requires Technical Assistants to examine and assess the strength of existing girder bridges and structures, and to prepare site surveys; salary range £875/948. Interesting work in pleasant conditions with promotion on Merit; superannuation fund; reduced rates of travel and other concessions; five day week. Applications, giving age, qualifications and experience, to Chief Civil Engineer, British Railways, Western Region, Paddington Station, London, W.2.

WESTERN REGION OF BRITISH RAILWAYS require Engineering Assistants (capable of supervising Drawing Office staff) and Technical Assistants experienced in the design of reinforced concrete and steel bridges and other structures to fill vacancies in posts in the salary ranges £1,095/£1,150 and £875/£948 respectively. Interesting work in pleasant conditions with promotion on Merit; Superannuation fund; reduced rate of travel and other concessions; five day week. Applications, giving age, qualifications and experience, to Chief Civil Engineer, British Railways, Western Region, Paddington Station, London, W.2.

SITUATIONS VACANT

A vacancy exists for an experienced R.C. designer/detailer or detailer/draughtsman. Good prospects; Varied and interesting work; Luncheon vouchers; Five day week. Apply with particulars of experience and salary required to John F. Farquharson & Partners, 34, Queen Anne Street, London, W.1.

A vacancy exists for an R.C. Designer or an experienced Designer/Detailer and for an experienced Detailer/Draughtsman. Good prospects. Varied and interesting work. Luncheon Vouchers. Five day week. Apply with particulars of experience and salary required to John F. Farquharson & Partners, 34, Queen Anne Street, London, W.1.

AN ENGINEER is required as an Assistant to this Company's Structural Engineer. He will be engaged upon the design of timber structures in which field previous experience is not essential. Applicants should be Graduates or of equivalent standard and able to work with initiative. This appointment offers an opportunity for a young Engineer to gain useful experience in timber design including Shell Roof construction. A Pension Scheme is in operation and a five day working week. Apply H. Newsum, Sons & Co. Ltd., Carr Lane, Gainsborough, Lincs., marking your envelope "For the attention of the Secretary."

BYLANDER, Waddell & Partners have vacancies for designer/draughtsmen and draughtsmen with two years minimum experience of R.C. work. Modern office with pleasant working conditions and opportunity for advancement. Salary in accordance with experience. Apply 169, Wembley Park Drive, Wembley.

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DETAILERS with minimum two years' experience.

The senior positions are of responsibility and interest and all are permanent with opportunity for advancement.

Please write with full details of experience and salary required to

BYLANDER, WADDELL & PARTNERS,

169, Wembley Park Drive, Wembley,

Middlesex.

CLARKE, Nicholls & Marcel require for their new Bristol offices, designers, detailers and draughtsmen experienced in reinforced concrete. Excellent opportunities in an expanding organisation. Positions are pensionable and offer first-class experience for those studying for professional qualifications. Apply in writing to 10, Apsley Road, Clifton, Bristol, 8.

COMMERCIAL Assistant to General Manager of large firm of Structural Engineers required in South Wales area. Only applicants with wide experience in Structural Steel Engineering should apply. Pension scheme, five day week and canteen facilities. Box No. 9116, STRUCTURAL ENGINEER, 43a, Streatham Hill, S.W.2.

CONSULTING Engineers have vacancy for Engineer with working knowledge of Soil Mechanics practice and its application to all types of foundation design. Site experience of reinforced concrete construction desirable. Apply stating age, qualifications, full details of experience and names of past employers, to Box No. 9110, STRUCTURAL ENGINEER, 43a, Streatham Hill, S.W.2.

CONSULTING Engineer, Victoria, requires Senior and Junior Designer/Detailers. Opportunity will be given to visit sites whenever possible. Apply in writing giving particulars of experience or telephone Vigilant 5611 after 7 p.m., James E. Wardropper, 50, Belgrave Road, S.W.1.

CONSULTING Engineers require capable assistants in their London office. Minimum of four years' experience. Interesting and varied work on reinforced concrete and steel structures and general civil engineering work for projects at home and overseas. Five day week, luncheon vouchers. Apply giving full particulars to Maurice Nachshen & Partners, 58, Victoria Street, S.W.1.

CONSULTING Engineers require in their London office experienced R.C. designers and designer/detailers; also experienced detailers willing to learn design. Five day week, good salaries for the right people. Apply in writing to S. Zukas, Chief Engineer, John de Bremaeker & Partners, 3, Southampton Place, London, W.C.1.

CONSULTING Engineers in Westminster require Engineer to take charge of site investigations and soil mechanics Laboratory. The post offers excellent scope for a well qualified, energetic man. Full details to Box No. 9115, STRUCTURAL ENGINEER, 43a, Streatham Hill, S.W.2.

DESIGN/ENGINEER with at least five years' experience in reinforced concrete, structural steel and general building construction. The work is varied and offers scope to take responsibility for projects, but does not involve working outside London except for site visits. A vacancy also exists for an Assistant Engineer with three years' experience as above. Write stating age, experience and salary required to Donovan H. Lee & Partners, Consulting Engineers, 66, Victoria Street, S.W.1.

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Applications are invited from qualified engineers with a good knowledge of structural design in steel for permanent appointments to the staff of our Special Projects Department in Central London.

The Department is engaged in advanced work on high masts and towers but previous knowledge of these is not expected. Successful applicants would be accorded every facility for learning design techniques at present in use. They would be responsible for the full design of structures, including foundations, and work in a pleasant atmosphere of mutual assistance between designers on difficult or unusual problems.

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Write, stating all details including qualifications, age and salary required to :—

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E. J. COOK & Co. (Engineers) Ltd., require Design Engineers and designer/detailers for reinforced and prestressed concrete. Salaries in accordance to ability and experience. Five day week; Luncheon vouchers; Pension scheme. Write or telephone for application forms from, 54, South Side, Clapham Common, London, S.W.4. Tel.: MACaulay 5522.

E. J. COOK & Co. (Engineers) Ltd., have vacancy for designer/detailer in Structural Steelwork office, with opportunity to gain experience in reinforced concrete work. Salary according to experience and ability. Five day week; Luncheon vouchers; Pension scheme. Apply to, 54, South Side, Clapham Common, London, S.W.4. Tel.: MACaulay 5522.

ENGINEER required by Consultants for research and development of new methods of design and the use of new materials for structures of large spans and/or 4-storeys high, as well as traditional structural and civil engineering work. Pension scheme, luncheon vouchers and three to four weeks holiday. Please apply to B. H. Fisher, of A. M. Gear & Associates, 12, Manchester Square, W.1. Phone Hunter 0331.

FAR EAST. Head of Import/Engineering Department required for Far Eastern Branch of well-known British Agency House with world-wide interests. The department is principally engaged in importing engineering products. Applicants should have good experience of the sales side of engineering. The ideal age range is 26-30. Graduate or Associate Members of one of the Professional Institutions preferred, although H.N.C. holders would be considered. Terms of service include furnished accommodation at nominal rent, frequent U.K. leave and free passages, local leave allowances, free medical attention, contributory Retirement Schemes and outfit allowance. For married staff there is a generous school fees grants scheme. Remuneration will be discussed at interview. Box No. 9112, STRUCTURAL ENGINEER, 43a, Streatham Hill, S.W.2.

GRADUATE Engineers and designer draughtsmen of high calibre required by well known London Consulting Engineers. A high rate of earning available for those with ability. Resident Engineers also required. Write to Box 9105, STRUCTURAL ENGINEER, 43a, Streatham Hill, S.W.2.

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MALAYA. Leading British Merchant House requires Civil Engineer for the Consulting Engineers' Department of its Malayan Subsidiary Company. This department is concerned with the Engineering aspects of the management of Rubber and Palm Oil Plantation Companies. Applicants must be under 30 and preferably Graduate or Associate Members of the Institution of Civil Engineers, although holders of the Higher National Certificate with the right experience would be considered. Furnished accommodation is provided at nominal rent. Frequent paid U.K. leave with free passages. Medical Scheme. Contributory Retirement Schemes. Other terms will be discussed at interview. Box No. 9114, STRUCTURAL ENGINEER, 43a, Streatham Hill, S.W.2.

R.C. DESIGNER/Detailer required in Westminster drawing office of Civil Engineering Contractors. Permanent position, five day week, pension scheme, profit sharing bonus. Salary in accordance with qualifications or if preferred state salary required, experience, age, etc., to Box 9117 STRUCTURAL ENGINEER, 43a, Streatham Hill, S.W.2.

R.C. DESIGNER/detailers and detailers required in Hammersmith office of Consulting Engineers. High salaries and good prospects with interesting work. Five day week; Pension Scheme; Holiday arrangements honoured. Apply in confidence with full details of experience and salary required to Alan Marshall & Partners, Federal House, 2, Down Place, W.6. Tel. RIVerside 8771.